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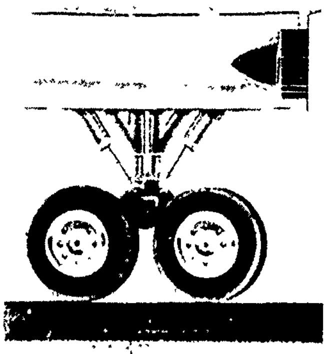

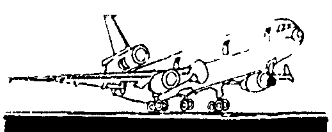

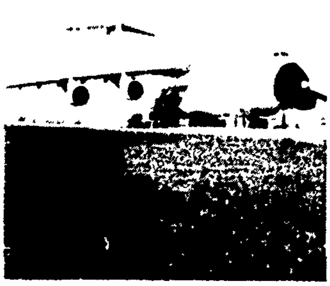

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ORIGIN OF DEVELOPMENTS FOR STRUCTURAL DESIGN OF PAVEMENTS

by

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Information has been compiled on the source or basis of the various aspects of Corps of Engineers design and evaluation criteria for pavements. An attempt has been made to convey understanding of the requirements, concepts, developments, impacting constraints, and unplanned occurrences relating to the assembling of pavement behavior knowledge for the 40 years following the initial concern beginning in 1940.

Individual coverage is provided, with pertinent interacting comment, for nine separate elements: (1) flexible pavement design methodology, (2) strength test and material characteristics for flexible pavements, (3) bituminous mix design and behavior, (4) rigid pavement design methodology including overlay design, (5) tests and materials for rigid pavements, (6) traffic and loading, (7) compaction requirements, (8) distress, failure, and terminal condition, and (9) concepts held and responses programmed.

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Compaction	Distress failure	Pavements	
Concepts	Evaluation	Rigid pavement	

PREFACE

This report of the background or history of Corps of Engineers pavement design methodology was sponsored by Headquarters, US Army Corps of Engineers (HQUSACE) under the Criteria Update Program. The Proponent and Monitor for HQUSACE was Mr. Paige E. Johnson, CEMP-ET.

The documentary effort was carried out by the US Army Engineer Waterways Experiment Station (WES), Geotechnical Laboratory (GL). Substantial input was arranged by contract with Mr. R. G. Ahlvin, an individual consultant uniquely qualified for such service. Mr. J. W. Hall, Jr. and staff of the Pavement Systems Division (PSD) provided guidance, support, and made available other facilities of GL and WES as necessary. General supervision was by Dr. W. F. Marcuson III, Chief, GL and Mr. H. H. Ulery, Jr., Chief, PSD, GL.

COL Larry B. Fulton, EN, was Commander and Director of WES. Dr. Robert W. Whalin was Technical Director.

CONTENTS

Page

Chapter 3. STRENGTH TESTS AND MATERIAL CHARACTERISTICS FOR FLEXIBLE PAVEMENTS

Chapter 4. BITUMINOUS MIX DESIGN AND BEHAVIOR

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*
feet	0.3048	metres
inches	2.54	centimetres
kips (force)	4.448222	kilonewtons
pounds (force)	4.448222	newtons
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic inch	27.6799	grams per cubic centimetre
square inches	6.4516	square centimetres

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

ORIGIN OF DEVELOPMENTS FOR STRUCTURAL DESIGN OF PAVEMENTS

CHAPTER 1

INTRODUCTION TO ORIGINS OF CORPS OF ENGINEERS PAVEMENT DESIGN

This report provides information on the source or basis of current Corps of Engineers design and evaluation criteria for pavements. It is an attempt to convey an understanding of the requirements, concepts, developments, impacting constraints, and occurrences over 40 years of concern for design improvements. It covers the time period 1940 to 1980. It is intended that this report will be supplemented at suitable intervals in the future.

During the early half of this 40 year period, the Corps had primary responsibility, as engineer for the Army, which included the Air Corps, and later as construction agent for the Air Force, for the design and evaluation of high performance and heavy-duty airfield pavements. From the late half of the period to the present the Corps has enjoyed close cooperation with Air Force civil engineering staffs and coordination with Navy pavement engineers, and has gained the opportunity to work with the Federal Aviation Administration (FAA) on mutual pavement concerns.

During World War II and following periods, the Military Construction Division of the Office, Chief of Engineers was supported by the Flexible Pavement Branch at US Army Engineer Waterways Experiment Station (WES), the Rigid Pavement Laboratory at the Ohio River Division Laboratory in Mariemont, Ohio, and the Frost Effects Laboratory at the New England Division Laboratory in Boston. The laboratories conducted research and other pertinent investigations and provided design manual criteria.

Concerns for flexible pavement design and related matters have been continued at WES to the present. Rigid pavement matters were transferred to the newly formed Construction Engineering Research Laboratory (CERL) in 1969. Beginning in 1971 all concerns for both flexible and rigid pavements, with the exception of pavement management systems (PMS) which were assigned to CERL, were combined at WES. Frost effect matters were moved to the newly formed Snow Ice and Permafrost Research Establishment, temporarily in available facilities on the outskirts of Boston (1940) but later moved to permanent

facilities in Skokie, Illinois (1945). Subsequently, the Corps interests in frost effects on pavements were again moved to a newly developed laboratory facility, the Cold Regions Research and Engineering Laboratory, near Dartmouth University in Hanover, New Hampshire.

The report is formatted to facilitate reference to specific items of concern. To this end, each of the following major elements of the total design system will be treated in a separate chapter.

Chapter 2.	Flexible Pavement Design Methodology
Chapter 3.	Strength Test and Material Characteristics for Flexible Pavement
Chapter 4.	Bituminous Mix Design and Behavior
Chapter 5.	Rigid Pavement Design Methodology Including Overlay Design
Chapter 6.	Tests and Materials for Rigid Pavements
Chapter 7.	Traffic and Loading
Chapter 8.	Compaction Requirements
Chapter 9.	Distress, Failure, Terminal Condition
Chapter 10.	Concepts Held and Responses Programmed

Each element is followed in its development from the initial impetus of World War II in the mid 1940's to recent times. Pertinent findings of research and experience are noted along with other influencing factors. Impact of the extant technology, then known as current concepts, and the basis of decisions shaping design criteria are discussed.

Presentations are generally chronological, with actual and approximate dates liberally included. It is often necessary to follow a particular element beyond initial concern for another element, with consequent overlapping and back tracking. Each section or subject area has its own bibliography with pertinent references following closely the order of presentation in the text. It is thus only necessary to follow dates of presentation and of references along with subject matter to select desired referenced work from the bibliographies. Specific references to bibliography entries are therefore not included in the text.

Because structural design methodology is the primary element of overall pavement design, and since concern for flexible pavement developments has remained, without interruption, at WES for the entire 40 years or more involved, the first section of Chapter 2 on "Flexible Pavement Design

Methodology" was developed first as a "pilot" or "pattern guide" for the other sections. Background and reference works were somewhat more available as were personnel familiar with the design developments.

CHAPTER 2

FLEXIBLE PAVEMENT DESIGN METHODOLOGY

The Corps of Engineers' concern for pavement design for heavier loadings began in November of 1940. Responsibility for design and construction of military airfields was then assigned to the Corps. War was threatening and the military air arm was still within the Army.

Selection of Design Method

A flexible pavement design method was required. Promising methods used bearing capacity of the subgrade as the basic design input, but means for determination of the bearing capacity remained in question. Initial studies and field investigations indicated that the time to develop independently a satisfactory test procedure was not compatible with the war emergency faced. Also, it was concluded that a plate-bearing test was not suited to military field needs nor to the assessment of subgrade shear which was considered of primary interest in flexible pavement behavior.

With recognition that a rational method for design, based on limiting stress-strain, was beyond reasonable promise within the foreseeable period of need, it was concluded that an established empirical highway design method should be adopted and further developed. It was considered that such an empirical method could be made to serve the pressing short-term needs. Far from abandoning concern for a rational design method, work on both theoretical and actual stress and strain induced in flexible pavements was planned to continue parallel with the more immediate functional method development.

Selection of CBR Method

Investigation of possible methods led to selection of California's CBR method for the following reasons:

a. The CBR method had been correlated with service behavior and construction methods (1928-1942).

b. The CBR method could more quickly be adapted to airfield pavement design for immediate use.

c. The method was thought to be as reasonable and as sound as any of the methods investigated.

d. Two states, other than California, had similar methods that had been successful.

e. The subgrade strength (CBR) could be assessed using simple portable test equipment in the laboratory or in the field.

f. Testing could be done on samples of soil in the condition representative of the foundation-moisture state commonly existing under pavements.

Single-Wheel Criteria

Criteria development began with the two California pavement design curves (1942) considered to provide for light and for medium-heavy highway traffic. These were taken to reasonably represent requirements for 7,000 lb* and for 12,000 lb wheel loads for airfield design.

Extrapolations were made by a board of consultants composed of the following:

a. T. A. Middlebrooks from Office, Chief of Engineers.

b. G. E. Bertram from Office, Chief of Engineers.

c. O. J. Porter, Developer of the CBR method.

d. Arthur Casagrande, world renowned soils consultant.

Curves were provided for 25,000, 40,000, and 70,000 lb wheel loads. At the time, the heaviest aircraft were the B-17 and the B-24 military bombers, and the larger B-29 was anticipated. Until this time, only single-wheel aircraft had existed.

Middlebrooks and Bertram, working together, made extrapolations based on equivalent values of maximum-shear-stress at pertinent depths for the various loads. Porter made extrapolations based on allowable deformation for the various loads. Casagrande's extrapolations were based on relationships between relative size of loaded areas. At this time, our mathematical models for stress, strain, and deflection extended only to a single uniform layer (homogeneous, isotopic, half-space) and only permitted "special case" (such as under the load center) determinations. Computations had to be made by slide rule or desk calculator.

The consultant board (1942), finding similar results for the three means of extrapolation and applying some consensus judgements, derived a set of CBR versus thickness curves for design of pavements for single-wheel aircraft for wheel loads from 4,000 to 70,000 lb. These, initially tentative, design curves were subject to verification by extensive accelerated traffic tests on

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page vii.

existing pavements of known composition and specially constructed full-scale test sections. With only minor adjustments in the high CBR range the curves were deemed to be satisfactory and were employed as engineer manual criteria from 1943 to 1949. During this time the design criteria existed as curves which could only be transferred by tracing or replotting values read from the curves. In anticipation of the B-36 aircraft employing single-wheel main gear, engineer manual revisions in 1946 added curves for 150,000-lb single-wheel loads. The Stockton No. 2 tests (1945-1948) included single-tire test loads of 150,000 and 200,000 lb for verification of the extrapolations to such large loads.

Multiple-Wheel Criteria

In 1945 and 1946 means were developed for using the established and verified single-wheel load design criteria to provide criteria for dual and dual-tandem (collectively multiple-wheel) landing gear loads. Studies of stress and deflection confirmed that at and near the surface individual wheels act essentially independently, while at substantial depths the overlapping of wheel-load effects results in much the same stress and deflection as would be induced by the total load on a single wheel. The depth at which a multiple-wheel load acted as only one wheel of the group was established at half the spacing between the edges of tire prints of the duals or the dual half of dual-tandems (d/2). The depth below which the total strut load acted as if it was applied on one wheel was established as twice the center-to-center distance between duals or the diagonally opposite wheels of dual tandems (2S). Between these depths the equivalent single-wheel load (ESWL) was represented by a straight line connection between the load on one wheel at a depth of d/2 and the entire strut load at a depth of 2S on a log-log plot of load versus depth. Determinations of stresses and deflections used for these developments were now possible by use of "Influence Charts" developed by Nathan M. Newmark (Newmark's Charts).

This means of determining ESWL was used for criteria formulation until verification tests in 1952 suggested a reexamination. The restudy, reported in 1955, led to development of the present method, which is based on equal deflection at each depth for theoretical deflections determined for a single

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Note: B-36 aircraft, except an early experimental version, were supported on dual-tandem wheel landing gears.

uniform layer. By this time the Boussinesq expressions for a point load on a uniform continuum had been integrated for a circular loaded area, and stress-strain deflection could be directly determined, though still by desk calculator or slide rule, for offset positions not under the center of the load.

The d/2 and 2S method for ESWL determination continues in use today (1990) by other organizations and nations. It can be used for a rapid means of approximating ESWL in relation to Corps of Engineers methods.

Influence of Tire Pressure

By 1947 the move toward multiple-wheel support of heavy aircraft was clearly established. Both the B-29 (dual)* and B-36 (dual-tandem)* aircraft had multiple-wheel gear. The aircraft designer's reluctance to accept weight increases also extended to the volumetric space for retracted gear. This led to use of smaller and higher pressure tires which technology could now provide.

It became necessary to adapt and extend the flexible pavement design method to accommodate such pressure increases. Accordingly, the existing pavement design curves (CBR versus depth for various loads), which were recognized as reflecting experience with tire pressures ranging from about 55 to 110 psi, were established in late 1947 as curves for up to 100-psi** tire pressure loadings. A following study, using "Theory of Elasticity" by Timoshenko for centerload deflections and "Newmark's Charts" for offset deflections (still calculator and slide rule), developed design curves for 200 psi and for 300-psi tire pressure based on equal deflections at the subgrade level based on a single uniform layer theoretical model.

In the 1940's a number of pavement behavior elements had gained sufficient definition to warrant criteria changes impacting on engineer manual guidance for the early 1950's.

Verification testing of the tire pressure extrapolations was accomplished in 1949. The testing confirmed the validity of the extrapolations. Criteria were formulated for 100- and 200-psi tire pressure single-wheel loadings.

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* During the 1960's, the Corps adopted a strong preference for use of twin and twin-tandem terms in place of dual and dual-tandem used earlier.

** Practice led to use of these criteria as 100-psi tire pressures curves.

Limited verification of the d/2 and 2S means for developing ESWL for dual and dual-tandem landing gear permitted formulation of design curves for B-29 and B-36 aircraft. The B-29 was a 100-psi tire pressure dual, and the B-36 was a dual-tandem having constant tire-contact areas of 267 sq in. The constant contact area was the result of the developing practice of inflating tires to an established rolling radius or a fixed tire deflection. Now, the B-50 was in existence. It was a dual of much the same size and weight of a B-29 but had tires the same as the B-36 and a constant contact area of 267 sq in. Design curves were formulated for the B-50.

Early CBR Equation

Up to this point the criteria existed as accepted curves which could only be copied by tracing or replotting. Now, however, the single wheel curves could be depicted as a set of curves by using equation 2-1:

$$t = k\sqrt{P} \qquad \text{(eq 2-1)}$$

where

t = thickness of structure

P = wheel load

k = a constant for a particular CBR and tire pressure

Values of k varied inversely with CBR and increased somewhat with tire pressure. The constancy of k was strong for CBR values below about 10. Now curves could be plotted from the equation, and established k values and interpolations and extrapolations were easily made.

The Stockton No. 2 results were also now available and established behavior for up to 150,000 and 200,000-lb single-wheel loads. These strengthened the advisability of accepting some of the heavier load (50 and 70 kips) adjustments necessary to adopt consistent k factors and equation 2-1.

Reduced Thickness in Runway Center Sections

By early 1949 some 7 years experience from existing airfields, accelerated traffic tests and direct experience during the Berlin air lift at Gatow, the British field, and the Rhein-Main airfield at Frankfurt indicated that runway ends, taxiways, and taxilanes on aprons were more critical than runway center sections. Direct observations, some measurement of vibration effects on pavements, and unpublished speed-deflection studies conducted by WES had eliminated recurring concern for impact loading in touchdown areas and

indicated that static to very slow loads were most critical. This was further confirmed by finding that dynamic (or rolling) loads produced smaller deflections than static loads. These observations led to introducing a 10 percent reduction of total structure thickness for runway center sections (between 1,000-ft ends). Recognition that aircraft are partially airborne at elevated speeds further supported the thickness reduction.

Operational Categories/Initial Load Repetitions Concerns

In the spring of 1949 a need developed for "Less than Capacity Operation" design curves for limited military use by the Air Force (now a separate service). A resulting study of traffic causing failure in accelerated traffic tests in relation to structure thickness indicated that limited traffic could be applied to pavements thinner than those established for normal design purposes. Prior to this, it had been accepted that a pavement structure which could sustain a substantial amount of traffic (this appears to have been in the range of perhaps 1,500 to 3,500 coverages* of wheel passes) would continue to support satisfactorily any reasonable amount of traffic. Even after the study demonstrating that thinner than standard design pavements could sustain limited traffic, the "Full Operational and Special Airdromes" category for "Theater-of-Operations" use (nominal 2,000 coverages) was considered to require the full thickness of regular designs (nominal 5,000 coverages) in the "Zone-of-Interior". This appears to be the earliest formal recognition that load magnitude and repetitions interact for purposes of pavement design and evaluation.

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* Aircraft traffic, even while taxiing, does not tend to follow closely a single path such that each pass of an aircraft would apply a load repetition in that path. To simulate actual airfield traffic, the load-cart passes in accelerated traffic tests were caused to travel along controlled central and off-set paths so as to cover a width representing the width subjected to loading by using aircraft on in-service pavements. This led to the term "coverages" to represent the loadings accumulating in the maximum central position of distributed wheel-passes from using aircraft traffic. The pattern of reducing repetitions with increasing off-set from a central path was also simulated in accelerated traffic testing. Studied patterns of "wander" from a central position permit determination of the total traffic in relation to the central accumulation of wheel passes which are represented as coverages. The comparisons of total to central accumulations of passes permit determination of pass-per-coverage relations which are presently (1990) in common use.

Revised Equivalent Single-Wheel Load Criteria

The multiple-wheel tests of 1949-1950 (reported in 1952) found the tentative design methods (based on d/2 and 2S) to be reasonably satisfactory but a bit unconservative. So by 1953 a reanalysis of all available data was programmed. By this time it was possible to compute complete sets of stress, strain, and deflection for any depth and offset from beneath the center of load, but only for a homogeneous, isotopic, linear elastic, half-space model. Computation was still by desk calculator or slide rule. Equivalent single wheel load (ESWL) methods were tried based on vertical normal stress, on maximum shear stress, and on vertical deflection. The means adopted uses equivalent maximum vertical deflection comparing the combined effects of all wheels of a multiple-wheel gear to one wheel having the same contact area as one of the multiple wheels. Others have more commonly chosen the ESWL to have the same tire pressure as the multiple-wheels. For Corps purposes, however, the equal contact area was chosen to permit treating individual wheel effects in terms of numbers of radii of circular contact areas for depth and offset positions. This divorces the geometry involved from the variation in loaded area resulting from ESWL variation when pressure is constant and greatly reduced computational complexity.

The equal maximum deflection basis of ESWL determination for deriving multiple-wheel design and evaluation criteria for flexible pavements was still in use in 1990. The deflections involved are computed using uniform pressure on circular areas for an idealized single-layer elastic model. It has been recognized that the theoretical deflection basins attenuate with offset distance more slowly than do the actual pavement structures, so that the contribution of widely spaced wheels to the cumulative maximum is likely too great. For twin and twin-tandem, this is not serious, but for more complex (C-5, Boeing 747, etc.) gear configurations, the overestimation of ESWL can become significant.

Modified Thickness Equation/Flexible Pavement Design Equation

A 1956 report presents further developments to the flexible pavement design equation. This advances the earlier equations to a form treating tire (or contact) pressure and CBR directly as shown in equation 2-2.

$$t = \sqrt{P\left(\frac{1}{8.1\ \overline{CBR}} - \frac{1}{p\pi}\right)} \qquad \text{(eq 2-2)}$$

or

$$t = \sqrt{\frac{P}{8.1\ \overline{CBR}} - \frac{A}{\pi}}$$

where

t = thickness

P = wheel load or ESWL

$\overline{\text{CBR}}$ = the strength test rating value

p = tire or contact pressure

A = tire contact area

This remains as the present form of the basic CBR equation, but in 1956 it represented a 20-year life design without provision for variation in stress repetitions (coverage level).

Traffic Repetitions/Coverages Introduced into the CBR Equation

No specific date can be related to acceptance of stress-repetitions (treated basically as coverages) as an element of design and evaluation applicable to the critical or design load. As earlier noted, the need for "Less than Capacity Operation" design curves for military application inspired a 1949 study showing the trend of required thickness versus coverages. In the following years the "Capacity" or "Zone of Interior" (ZI) design criteria were accepted as representing a 5,000 coverage use life (nominally 20 years). By the mid 1950's the channelized traffic problem was experienced, studied, and treated. This represented proof for developing concepts arguing against the long held idea that a pavement satisfactory for a few thousand repetitions should be satisfactory for many thousand repetitions.

One of the curves from the 1949 work which related percent-of-design-thickness to coverages of critical load had become accepted with 5,000 coverages at 100 percent design thickness. The curve was stated mathematically about 1960 as follows:

$$\%t\left[\frac{1}{100}\right] = 0.23\ Log\ C + 0.15 \qquad \text{(eq 2-3)}$$

$$(\%t = 100 \ at \ C = 5{,}000)$$

where C equals the coverages, and the log is the common or base 10 log. This leads to the more general CBR equation:

$$t = (0.23 \ Log \ C + 0.15)\sqrt{P\left(\frac{1}{8.1 \ CBR} - \frac{1}{p\pi}\right)} \qquad \text{(eq 2-4)}$$

More details on stress repetitions are presented in this report.

Channelized Traffic

The "Channelized Traffic" problem had a strong impact on flexible pavement design for heavy aircraft, and while this impact did not result in enduring permanent effects on design and evaluation criteria (excepting effects on traffic distribution), the interim adjustments to criteria and concerns at the time warrant some examination.

By 1953 the B-47 aircraft was replacing the B-36 as the primary Air Force bomber type, and by early 1955 a number of flexible pavement airfields sustaining B-47 traffic were experiencing unexpected distress. This was commonly a grooving of straight sections of primary taxiways along their center lines. It was found that a number of factors were combining which resulted in a dramatic increase in the rate of application of coverages in critical areas. The B-47, having bicycle gear, applied two gear passes for each aircraft pass. The practice of painting taxi-stripes for pilots to follow and steerable nose gear permitting better guidance narrowed the aircraft lateral wander and increased the coverages for the passes experienced. But the primary factor was the ease of preparing the B-47 for flight. Preflight activities could be accomplished in only a few hours while the B-36 had required days. As a result, the B-47's were flying many more times per aircraft per year.

In response to the problem and preliminary examination of effects, the Corps, in June 1955, issued Interim Design Criteria for Airfield Pavements to be Subjected to Channelized Traffic of Heavy Aircraft. These criteria increased total thickness requirements about 25 percent, increased base course thicknesses, and increased subgrade compaction requirements. It also introduced a requirement for proof rolling with a heavy rubber-tired roller on top of the subbase and on each layer of base course placed. For details on

compaction requirement developments, one can refer to the later presentation herein.

Studies were undertaken to determine aircraft wander from a central position on taxiing (1956), to investigate behavior of pavements subject to channelized traffic of heavy aircraft on existing airfields (1960), and to conduct accelerated traffic tests (1962) to verify or revise the "Interim Design Criteria" regarding increased requirements for thickness and compaction.

These studies showed that on taxiing the dual gear of B-47 aircraft or similar gear of KC-97 aircraft stayed within a lane only 7-1/2 ft wide 80 percent of the time. It was found that load repetitions (coverages) were being applied at a rate of about six times that considered in existing design criteria (5,000 coverage life). A 20- to 25-year life pavement was exhibiting terminal distress in only 4 years or less. On completion and analysis of the accelerated traffic testing, along with field studies of effects of channelization of traffic, it was concluded that the problem was largely one of densification of subgrade under the increased load repetitions and insufficient compaction requirements. Later discussion contains details of compaction criteria.

The 25 percent increase in thickness requirements established by the "Interim Criteria" (1955) was subsequently not found to be required, yet some increase in structure (thickness) requirements could not be ruled unnecessary. It was concluded that the increase in thickness, consistent with increase coverages from 5,000 to 30,000 (later reduced to 25,000-see the section on load repetitions) would reasonably respond to the structure strengthening need. Detail on such matters was about 1962. The part of the CBR equation relating to repetitions could be applied as:

$$\%t\left(\frac{1}{100}\right) = 0.23\ Log\ C + 0.15 \qquad \text{(eq 2-5)}$$

Porpoising of Bicycle Gear Aircraft

With the introduction of bicycle-gear aircraft (1952-1953) came another problem of concern. The B-47 type aircraft experience a new and different response to bumps and grade changes. The phenomena, commonly described as porpoising, was a longitudinal rocking of the aircraft. A bump successively encountered by front and rear gear at a speed producing load (or force) input

at near resonant response of the aircraft resulted in severe (front or rear) rocking or "porpoising". Where two or more bumps or grade changes were encountered at critical spacing, the aircraft was further excited in resonant response.

Pilot complaints included severe shaking, inability to read instruments, damage to instruments, loss of control of the aircraft, and aircraft becoming airborne prematurely. At the time (about 1953) there was great concern for the possible impacts of this phenomena on pavement requirements.

It was found that severity of porpoising did not relate to roughness magnitude, but appeared to relate somewhat to spacing of maximum deviations from grade. Concern for the problem did lead to a tightening of smoothness requirements. However, by mid 1957 a reported study of porpoising indicated that corrective measures appeared to have taken care of the problem. Boeing had studied and revised shock strut damping for the B-47, and the problem for B-52 aircraft was not significant.

With the advent of the porpoising problem, and likely as a result of it, concern developed for the dynamic response of aircraft to runway profiles and roughness. This extended to both the increased dynamic pavement loadings and the pavement induced dynamic stresses in the aircraft. Mathematical developments had matured sufficiently to permit some treatment of the problem, but computer capability was not yet available. Study of dynamic response of aircraft to runway surface configuration has continued. Rapid means for roughness and profile assessment have been developed and others have been attempted or proposed. Computer modeling of aircraft response to runway profiles has made great strides.

Bicycle Gear Factor

In the mid 1950's because of unknown aspects of channelized traffic, of porpoising, of following a front gear loading with the second rear gear loading without allowing recovery time, and of recognizing that transport and bomber type aircraft grow in gross weight in the years following the first appearance, a "bicycle gear factor" was introduced into the design criteria for B-52 and B-47 aircraft pavements. This introduction was accomplished by employing the thickness criteria for a 275,000 lb B-52 gear loading as criteria for a 240,000 lb B-52 gear loading. Other B-52 and B-47 criteria were adjusted proportionately (275/240). This is a nominal 15 percent

increase in load support requirements as a safety factor against unknown and unforeseen elements.

Proof Tests

Some comment on the Kelly and Columbus Air Force Base (AFB) "Proof Tests" conducted in 1956 and 1958 is pertinent at this point. The tests were largely structural and applicable to both flexible and rigid type pavements, but their impact was significant in relation to asphalt mix design. The comments in the section on "Bituminous Design and Behavior" may also be of interest.

An Air Force policy that required rigid pavement in certain critical areas of airfields and permitted a 5 percent cost premium for rigid pavement in other areas led to a congressional inquiry on the matter in early 1954. There was then no demonstrated difference in structural behavior or in maintenance requirements for the two pavement types. Investigations and observed field performance had established that asphalt concrete pavement was less resistant to fuel spillage and jet blast than portland cement concrete (PCC) pavement. The Congressional subcommittee concurred in the Air Force classification of aprons and 1,000 ft runway ends as critical areas and in requirements for these to be PCC. The subcommittee did not concur in the 5 percent premium for PCC in other areas.

In August 1954 the Air Force introduced these concepts into criteria for pavement type selection and restated the desire for PCC in all pavements. By late 1955 the channelized traffic and porpoising problems were impacting, and in December 1955 the Air Force specified that all pavements on which aircraft are normally operated, parked, serviced, or maintained should be classed as primary use pavements to be constructed of PCC. By now both flexible and rigid structural criteria had been modified to accommodate the channelized traffic of B-47 aircraft. A proof test program was undertaken to establish the validity of the modified criteria and to determine the ability of contractors to carry out the more demanding specification requirements.

The tests were conducted at Kelly AFB, Texas in 1956. Results established were as follows:

a. The rigid pavement design and construction procedures developed for channelized traffic of B-47 aircraft were validated.

b. The flexible pavement design and construction procedures for total thickness and compaction requirements for the channelized traffic of B-47 aircraft were validated.

c. The design and construction procedures for the asphalt concrete portion of the flexible pavement did not produce pavement capable of withstanding the hot-weather traffic conditions imposed.

Directly after the Kelly tests, the Corps procedures for asphalt concrete mix design were revised to accommodate the increased coverages and increased tire pressures.

More Congressional hearings (1957) followed questions of the fairness of the application of all 30,000 coverages (10 to 20 years of traffic) at temperatures above 90°F. It was agreed that about 1/3 or 10,000 coverages of hot-weathered traffic would be more realistic. Therefore, further tests were programmed at Columbus AFB, Mississippi. These were intended primarily to assess the capacity of flexible pavement to sustain B-52 traffic in runway interiors (noncritical areas), but a transition design for the joining of flexible and rigid pavement was included, and the rigid pavement was trafficked.

Conclusions drawn from the testing were as follows:

a. The pavements were designed and constructed under normal contract conditions.

b. Considering normal B-52 operations, the tests at Columbus AFB demonstrated the validity of the design and construction procedures developed for heavy-load flexible pavements for runway interiors.

These conclusions were presented to the investigating Congressional subcommittee in December 1958. In a March 1959 report the subcommittee recommended that the engineering conclusions presented in the December 1958 report be accepted, adopted, and implemented by the Air Force, the Corps, and the Bureau of Yards and Docks. The subcommittee also recommended that future bids for pavement in areas where both pavement types have proved satisfactory employ alternate options for pavement type.

<u>Verification of Thickness/CBR Equation</u>

Following up on the 1956 CBR equation developments, which had resulted in equation 2-6:

$$t = \sqrt{P\left(\frac{1}{8.1\ CBR} - \frac{1}{p\pi}\right)} \qquad \text{(eq 2-6)}$$

or

$$t = \sqrt{\frac{P}{8.1\ \overline{CBR}} - \frac{A}{\pi}}$$

and making use of the newer (1955) equivalent deflection method for ESWL, a further verification of the expanded equation was undertaken. Parameters in the equation were thickness, wheel load, CBR, and contact area or pressure. But using $P = A \times p$, these can be reduced to only two variables:

$$t = \sqrt{\frac{A(p)}{8.1\ \overline{CBR}} - \frac{A}{\pi}} \qquad \text{(eq 2-7)}$$

or

$$\frac{t}{\sqrt{A}} = \sqrt{\frac{1}{8.1}\ \frac{1}{\frac{\overline{CBR}}{p}} - \frac{1}{\pi}}$$

The two variables are:

$$\frac{t}{\sqrt{A}} \text{ and } \frac{\overline{CBR}}{p}$$

By converting multiple-wheel loads to pertinent ESWL's, it was possible to place all available experience type data from accelerated traffic tests and recorded field behavior experiences on a single plot of $t/\sqrt{A}$ versus CBR/p . Also, the CBR equation curve, a single curve using these parameters, could be placed on the plot for comparison. The result was a remarkably strong pattern of separation of failed and nonfailed, test and prototype pavement points by the CBR equation curve.

The reporting of this strong correlation in early 1959 gave great support to the validity of the basic CBR equation. It was still, however, not common practice to relate behavior to stress repetitions (coverages) as well as loads. Satisfactory (nonfailure) performance was considered to be established by 2,000 or more coverages. The correlation and basic CBR equation represented a substantial step toward rationalization of the CBR system.

CBR Design Curve Formulation

This rationalization of CBR design, rather than involving a series of separate contributions, was a continuing process extending from the first equation development ($t = k\sqrt{P}$) in 1949 and adjustment to its pattern, through the multiple-wheel (ESWL) developments of 1955, the expanded CBR equation of 1956, the combined criteria analysis plots of CBR/p versus $t/\sqrt{A}$ of early 1959, and the November 1959 Instruction Report No. 4, "Developing A Set of CBR Design Curves."

Instruction Report for Determining Thickness Requirement

Instruction Report No. 4 presented the basic CBR equation and the curve it represents as a plot of CBR/p versus $t/\sqrt{A}$. The report explained adjustments for the higher CBR range. It presented the method for determining ESWL for any wheel assembly. It also included the (by now accepted) plot of coverages (log scale) versus percent of (5,000 coverage) design thickness. This relation had not yet been formally documented in its equation form (0.23 log C + 0.15), but this was soon to follow. The result was the full CBR equation (1960 and beyond):

$$t = (0.23 \, Log \, C + 0.15)\sqrt{\frac{P}{8.1 \, CBR} - \frac{A}{\pi}} \qquad \text{(eq 2-8)}$$

C = coverages

Pavement Behavior Concepts/Corps CBR Design Procedures

Through the early and intermediate years (1940's, 1950's, and 1960's) during which the CBR procedures were developed, interdependent program elements were planned and behavior concepts developed by those responsible for guiding and contributing to the developments. These were discussed in a January 1958 paper, Miscellaneous Paper 4-252, "Notes on the Corps of Engineers' CBR Design Procedures." An examination of these program elements and behavior concepts will be valuable to aid in understanding the background and basis of current Corps design criteria.

Design Concepts

Flexible pavement design, in broad concept, embodies two features which deal with the pavement structure and a third which deals with the bituminous mixture.

a. Each layer must be thick enough to distribute the stresses induced by traffic resulting in a stress level which will not overstress and produce shear deformation in the next underlying layer. The CBR procedures are intended to indicate the thickness of overlying structure required to prevent shear deformation in any layer of the structure. Note that thickness design by the Corps CBR method is applicable to each layer of the structure and to soft or uncompacted layers below the subgrade surface. Many CBR users consider the method applicable only to total structure thickness above the subgrade. The emphasis on shearing and shear deformation should be noted as the element of concern in thickness design for load support. Later developments (the 1970's), particularly in the highway field where stress repetitions are an order-of-magnitude greater than for airfields, have placed emphasis on fatigue cracking of asphalt surfacings. This has detracted from the earlier, uniquely and strongly held concept that resistance to shearing is the prime concern for pavement strength design.

b. Each layer must be compacted adequately so that traffic does not produce an intolerable amount of added compaction. This is a consideration separate from that of preventing shear movements. An increase in density caused by traffic, even though it results in undesirable surface rutting, represents an increase in strength or resistance to shearing. Surface rutting from internal shear movements represents a decrease in strength of the layer or layers being sheared.

c. The flexible pavement must have a wear- and weather-resistant medium as a surface that will not displace under traffic.

Thickness Design Concepts

Only the first of these three elements is of concern in this section. Compaction design and bituminous mix design are treated in later sections. The Miscellaneous Paper 4-252 notes that thickness design has two basic parts: (1) determining the protective thickness required for a soil (material) with a given CBR value, and (2) estimating the CBR the soil will develop after it has been placed in the pavement system and the moisture content has become adjusted to the weakest seasonal and/or long-term condition. This concept of

designing for the poorest condition which could be predicted as likely to exist in a pavement during its use-life was virtually universally held and unquestioned in all early work. The only exceptions were in relation to freeze-thaw considerations, wherein an option of limiting load or use during periods of thawing was sometimes applied to protect a pavement. Aspects of this minimum strength design concept continue to the present, but in the last 15 or 20 years there has been growing recognition of the impact of periods of other-than-minimum conditions on considerations of cumulative (equivalent critical) stress repetitions on pavement life projections.

Actions to Improve Criteria

Miscellaneous Paper 4-252 mentions various supporting elements of the broad program pursued to devise, to strengthen, and to eventually supplant with more rational methods the CBR based concepts initially selected.

a. Program plans and accomplishments were subjected to regular periodic review by consultant boards concerning thickness design, compaction criteria, bituminous surfacing, and more rational methods.

b. To better understand how to project the minimum strengths to be expected of pavements a "Field Moisture Study" program was undertaken. A series of selected in-service airfields across the southern part of the United State were subjected to regular seasonal examination for a number of years to determine the pattern of moisture and density existing within the pavement layers as affected by environmental variations. It was these studies that showed the "soaked-CBR" test to well reflect strengths to be expected from more plastic materials and to only somewhat conservatively reflect strengths of less plastic materials. It was also these studies which showed no significant seasonal variation in moisture and strength conditions beneath (sound) wide pavements except for perhaps 15 or 20 ft at the edges.

c. Recognizing that verification of the validity of design must come from in-service behavior, the Corps laboratories undertook a regular program of condition survey and evaluation of US Air Force pavements to compare performance with design concepts. This was extended to Army airfield pavements as concern for these developed in about 1959-1960.

d. When, at the outset (early 1940's), a fundamental basis for pavement design had to be given up in favor of a functional empirical method, a program for theoretical design developments was undertaken. These included a study of analytical methods, idealized full-scale field test sections designed to

permit measurement of stress, strain, and deflection consistent with then available theoretically derived values, and development of instruments and methods for field measurements of stress, strain, and deflection.

Criteria for Roads and Streets

In 1961 thickness design criteria were developed for roads and streets consistent with the existing technology. Prior Corps criteria were based on experience and practice, were extremely simplistic, and had not been updated for a long time. While the emphasis of this report is strongly focused on airfields, the reference to road and street design is justified because it introduces significant aspects of load repetitions.

The CBR equation had by then gained the common form of:

$$t = f\sqrt{\frac{P}{8.1\ CBR} - \frac{A}{\pi}} \qquad \text{(eq 2-9)}$$

where f was the load repetitions factor (used as a decimal fraction) taken from the plot of percent of (5,000 coverage) design thickness versus coverages. The factor f was being employed directly in the form:

$$f = 0.23 \log C + 0.15$$

thus

$$t = (0.23\ Log\ C + 0.15\sqrt{\frac{P}{8.1\ CBR} - \frac{A}{\pi}} \qquad \text{(eq 2-10)}$$

but this form of the equation had not enjoyed common publication in WES reports.

The 1961 publication of revised road and street design criteria (Technical Report 3-582) presents a plot of percent design thickness versus coverages which has been extended to 10^7 coverages. Technical Report 3-582 presents, for the first time, equivalent operations factors (in terms of 18,000-lb single-axle loads), shows pass-per-coverage developments, and explains means of combining the effects of an array of vehicle loadings into a single magnitude of 18-kip single-axle load equivalents.

Mixed Traffic

The practice of expressing mixed traffic effects on roads in terms of equivalent 18-kips axle loads was soon broadly adopted as an outgrowth of presentations in the first "AASHO Interim Guide" put out by an AASHO committee suggesting means of application of the AASHO road test results.

Mathematical and Computer Developments

By the late 1950's and into the 1960's mathematical and theoretical developments supported by computer developments available at the IBM Watson Laboratory and in various larger universities had permitted the publication of tabular stress, strain, and deflection values for 2- and 3-layer analytical models based on the theory of elasticity. By the later 1960's n-layer elastic models for use on more widely available (large but not yet user-friendly) computers were being applied in pavement design by the leading researchers and designers. Developments were leading to finite element methods and to stress-dependent-moduli. Limited computer capacity, however, was still greatly limiting the pursuit of more comprehensive models.

Initial NDT Developments

In the mid 1960's work began on what later became the broad interest in nondestructive testing of pavements. This began with introduction of steady-state vibratory loading of pavements and the study of pavement response. Initially there was prime interest in measurement of wave propagation from a vibratory loading, both wave length and wave velocity. These measurements permitted an assessment of the modulus of elasticity of pavement layers below the surface. It also resulted in thickness-of-layer determinations as one-half the wave length of waves induced in each layer, but this was a crude and not easily analyzed determination now largely abandoned.

The wave propagation work did lead to the relation between E modulus and CBR which has commonly been employed in later NDT work when no better basis was available.

$$E \text{ (in psi)} = 1{,}500 \text{ CBR} \qquad \text{(eq 2-11)}$$

The direct assessment of E from wave velocity has been employed for pavement subgrades and for foundations for dynamic design.

Modifications due to Very Large Aircraft

By 1970 the Jumbo-Jets (later wide-body aircraft) had arrived (C-5 military and 747 civil) at weights going well beyond a half million pounds. These

aircraft brought new landing gear configurations and a need to consider the interaction effects from more than four wheels.

To verify greatly extrapolated design and evaluation criteria, multiple-wheel heavy gear load (MWHGL) tests were carried out at WES. These included rigid as well as flexible pavements and test traffic representing both the C-5 and Boeing 747 aircraft.

New Form of CBR Equation

Changes were introduced into the design criteria for flexible pavements to reflect findings of the MWHGL tests. The basic portion of the CBR equation, as represented by the curve of Instruction Report No. 4 plotting

$$\frac{CBR}{p} \text{ versus } \frac{t}{\sqrt{A}} \qquad \text{(eq 2-12)}$$

was retained, but a third degree form of the equation which had been formulated was used:

$$\frac{t}{\sqrt{A}} = \alpha - 0.0481 - 1.1562\left\{Log\frac{CBR}{p}\right\} - 0.6414\left\{Log\frac{CBR}{p}\right\}^2 - 0.4730\left\{Log\frac{CBR}{p}\right\}^3 \qquad \text{(eq 2-13)}$$

The portion of the CBR equation which adjusted thickness for other than 5,000 coverages (0.23 log C + 0.15) was supplanted by an α factor. Thus the equation became:

$$\frac{t}{\sqrt{(A)}} = \alpha\left[-0.0481 - 1.1562\left\{Log\frac{CBR}{p}\right\} - 0.6414\left\{Log\frac{CBR}{p}\right\}^2 - 0.4730\left\{Log\frac{CBR}{p}\right\}^3\right] \qquad \text{(eq 2-14)}$$

The equation and the α - factor curves are presented in the 1971 MWHGL reports, Miscellaneous Paper S-71-5 and Technical Report S-71-17.

Pavement Deflection

A long recognized pavement behavior phenomena of significance in relation to the background of pavement design was formally treated in a 1971 report, "Deflection-Coverage Relationship for Flexible Pavements", Miscellaneous Paper S-71-18. This study showed a strong relation between elastic (or recoverable) deflection and allowable load repetitions on flexible pavements.

Since the mid 1950's, in both highway and airfield studies, there have been reports using deflection of flexible pavement under load providing an indication of the allowable load magnitude for that pavement. In accelerated traffic testing at WES it had become recognized that a pavement load which would cause an elastic deflection of about 1/4 in. could be expected to fail the pavement with repeated loading in excess of 2,000 coverages. For some wide-tire low pressure loads, the limiting deflection might exceed 1/3 in. For some narrow-tire high pressure loads, the limiting deflection might be less than 0.15 in.

In highway response work the reported limiting deflections were in the range of less than 1/10 in., and some were as low as 0.02 to 0.03 in. AASHO Road Test data showed fairly wide ranging deflections but values averaged about 0.1 in. for items considered terminal after 30,000 to 40,000 repetitions and about 0.06 in. for items lasting to 200,000 to 300,000 repetitions.

Studies at WES through the years had provided indications of low repetitions to failure of test pavements (severe overload) showing larger than the nominal 1/4-in. deflection under load. It should be cautioned here that there has been a tendency for misunderstanding of these deflection criteria. It has been easy and incorrect to consider that after application of some number of coverages the pavement will show the elastic deflection indicated by a correlation of deflection with coverages. The critical elastic deflection values recognized are indicators of the total life of the pavement as it is subjected to repeated applications of the load which caused the deflection. The deflection under load tends to deflect the same magnitude whether a pavement is new (following any initial adjustments it may sustain) or is nearing (but not yet seriously deteriorated by) the end of its useful life.

The study reported in Miscellaneous Paper S-71-18 in 1971 on deflection-coverage relationships had been suggested many times in earlier years, and was finally carried out. Impetus was its significance in relation to nondestructive testing methods being developed. The final correlation

presented ranges from below 10 coverages to above a million coverages. It is a very strong pattern but has an undesirable spread in the data.

Later NDT Developments

Through the early years and into the mid 1970's there was strong interest, both within the Corps and in the pavement technical community outside the Corps in nondestructive testing of pavements. Interests had focused away from wave velocity determinations and had concentrated on the deflection response of pavements to repetitive (steady-state) dynamic loading and most recently to impluse loads. The dynamic stiffness modulus (DSM) was introduced and correlated with expected behavior. The DSM is the ratio of load to deflection or slope of the load-deflection curve using a particular steady state vibrational load test. Standardized procedures were devised for flexible pavements for the US Air Force, the Federal Aviation Administration, and the Army.

These methods were all based primarily on the use of the Corps "16-kip vibrator," a system mounted in an 18-wheel tractor-trailer combination. An electrohydraulic vibrator applies steady-state vibratory loads of up to 15 kips (30,000 lb peak to peak) onto pavements, and the responding deflections are measured at a range of loadings up to 15 kips. From this the DSM is determined and used for evaluation of the pavement's load-support capacity.

Emphasis has subsequently (late 1970's) shifted to use of falling weight deflectometers (FWD) for dynamic pavement loading. Deflections are measured beneath the load and at various offset locations to determine shape and magnitude of the "deflection basin" resulting from loading. By this time layered-elastic analysis methods were in wide use, and widely available personal computers (PC's) have made applications common.

Layer stiffnesses can be back calculated using the deflection basin measurements, and stresses and strains can be calculated for any location within the pavement system. Correlations of flexural strain magnitudes in the surface layer and vertical strain magnitudes in the subgrade have been established in relation to load repetitions and magnitude. These are being used for pavement evaluation.

Structural Layers and Equivalencies

In the early to mid 1970's provision was made in design for the superior load distributing capability of "structural layers" in pavement systems, as developed by the AASHO road test analyses some 10 years earlier. It became the practice in highway work generally to recognize "equivalency factors"

applicable to pavement layers stronger than required for load spread as a function of thickness and resistance directly to induced shearing. This recognized a superior load spread through flexural (beam-action) strength.

Primary focus was on stabilized layers, but subsequently, the superior behavior of better quality granular materials was also treated.

As early as 1954, engineers had recognized this possible phenomena and had attempted to study the effect using data from the Stockton No. 2 tests (1948). These tests included items of equal thickness which were made up of different thicknesses of bituminous bound materials, base quality materials, and subbase quality materials. This analysis clearly indicated that at elevated temperatures the bituminous bound pavement layers were not superior in load distributing capability to excellent quality (100 CBR) base materials. There were indications, but insufficient comparisons for conclusion, that base quality material placed where only subbase quality was needed would yield superior performance. However, the indications, though insufficient were that the extra advantage of the better than needed materials was only slight, a few percentage points at most and not the 1-1/2 to 2 or 1 to 2 being adopted in highway work.

These early conclusions were a restraint on Corps acceptance of the highway industry trend toward use of substantial equivalency factors for design and evaluation of flexible pavements.

By 1974, however, full-scale field tests were undertaken at WES, which resulted in adoption by 1977 of equivalency factors for Corps design and evaluation purposes.

<u>All Bituminous Concrete Pavements</u>

The mid 1970's also saw a Corps response to another area of interest which had developed in the flexible pavement design community. Concern for the flexural behavior of surface layers, in better load distribution, along with the lower tensile strains in thicker asphalt layers with limiting strain which is now a consideration in layered system analyses, led to interest in much thicker bituminous layers for pavements. This was further advanced by the finding or recognition that effects of temperature gradients on compactibility and the gradients of compacting effects beneath a roller were complementary, so that good density throughout much thicker bituminous layers can be readily attained. Corps response to this was a 1975 report, "Development of a Structural Design Procedure for All Bituminous Concrete Pavements

for Military Roads." This also represented a first formalization of a Corps design based on layered-elastic analysis.

Dynamic Effects of Aircraft

Continuing interest in dynamic effects of pavements on aircraft and aircraft on pavements, emphasized by convictions of some leading pavement engineers that intermediate range jet aircraft were affecting pavement behavior in a hitherto unrecognized manner, led to further studies by the Corps in concert with the FAA. These were studies involving instrumented aircraft and instrumented pavements conducted and reported in 1975 and 1976.

While a broader awareness of the interaction of aircraft and pavements was gained, no significant design modifications or trends resulted. It has been generally recognized that intensified distress resulted as some of the intermediate range jet aircraft (particularly the B-727 but DC-9 also) replaced earlier propeller aircraft. The intensified distress was the result of a capability of these aircraft to use airfields which were too short for earlier types of comparable weight (an overloading problem and not a new dynamic effect).

Layered-Elastic Analysis

In 1975 the Corps also developed a layered-elastic design procedure for flexible airport pavements. This was, at the time, presented as an optional design method for heavy flexible pavements.

Updated Instruction Report

In mid 1977 an upgraded version of guidance (Miscellaneous Paper S-77-1) toward the development of CBR design curves was published. This covered treatments of ESWL, pass-per-coverage relationships, the newer α curves, and the CBR equation in both old and newer forms.

Recent Developments

Impact of the technology of the 1980's on Corps pavement design criteria will not be characterized at this time (1990). Pertinent developments and references are still commonly available, and more time needs to pass before a backward look will permit generalized characterization of significant effects on design.

Flexible Pavement Design Methodology

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CHAPTER 3
STRENGTH TESTS AND MATERIAL CHARACTERISTICS FOR FLEXIBLE PAVEMENT

Introduction

With adoption of the CBR design methodology in 1942 there was a need to become familiar with and improve the basic strength test on which the method is based. In November 1942 a comprehensive laboratory study of the CBR (California Bearing Ratio) test was begun. The study was completed and reported in July 1945 in TM 213-1.

Comprehensive Laboratory Study

The California practice (1940-1942) was to conduct a moisture-density test using its 6-in. diameter mold specimens approximately 5 in. high and static compaction to determine optimum moisture content. For the CBR determination, a remolded specimen was prepared at optimum moisture, soaked 4 days confined by a 12-1/4 lb surcharge weight with soaking water available to both top and bottom of the specimen, and penetrated using a 3-sq in. end area (round) piston. Compaction for both the moisture-density test and the CBR determination was accomplished by application of a 2,000-psi static load in the 6-in. diameter mold.

California used the 2,000-psi static load in both laboratory and field prepared specimens, but for Corps purposes, it was recognized that the static load would be impractical for field use, especially in military applications. Even before beginning the comprehensive laboratory study of CBR in November 1942, the Corps developed the modified density test which has become standard and is variously designated as Modified AASHO, Modified Proctor, or CE-55 compaction. The modification involved increasing hammer weight from 5-1/2 to 10 lb, drop height from 12 to 18 in., and soil layers from 3 to 5. The modified compaction test first appeared as a construction control test in the June 1942 "Engineering Manual for War Department Construction."

Moisture Density

In 1942 it was considered that representing field moisture and density in a laboratory specimen would be sufficient to have a test specimen of the same strength (shear resistance) as that developed in the field. There was then complete concensus that the CBR test is a penetration shear test used to determine a modulus of the shearing resistance of soils.

The CBR test program began in November 1942 and reported in July 1945 showed that merely matching moisture and density does not provide a specimen of matching strength. There are elements of soil structure, later recognized as patterns of particle orientation, which must be matched also to better represent strength of construction in prepared test specimens. Effects of static versus dynamic compaction, wet side versus dry side compaction, and interactions of these in molding plus soaking all had significant impact on strength.

Laboratory Studies

The comprehensive laboratory study of the CBR test included extensive testing on 20 soils. The purpose was to determine the effect of certain variables on the CBR. Variables such as method of compaction, water content, density, specimen size, time of soaking, method of soaking, soaking surcharge, drainage time after soaking, penetration surcharge, rate of penetration, and effect of oversize particles were studied. Comparative unconfined compression and triaxial shear testing was included.

The study showed that variations in CBR results were largely explained by means of preparing test specimens. Variations were systematic and largely related to molding water content, density, and method of compaction. It was felt, and later substantiated by field test experience, that dynamic compaction of specimens better simulates field construction conditions than does static compaction. The soil structure aspects of simulating field strength in terms of CBR were found to be best represented by controlling molding water content and density than simulating near-saturation field conditions by soaking specimens. Unconfined and triaxial tests showed equivalent behavior.

Since the CBR test was being closely examined and many aspects of its conduct and application decided, prevailing concepts still were that a structure or element of the pavement structure must have some minimum strength to resist or support some maximum load. The concept of trade-off between load magnitude and load repetitions was not yet a consideration. There was also, as yet, no thought of some weighted or combining means for recognizing variation in load supporting strength with variation in subgrade or other structure element strength. The concept was one of determining the minimum strength expected to be obtained and basing structure design on this minimum.

Reductions for Arid Regions

There was recognition that subgrades and subbases would, at some time, tend to increase moisture content approaching saturation except in quite arid situations. The degree to which soaking CBR specimens would represent prototype conditions was not known, but the decision was made to accept any conservatism. To provide some relief, early design manual guidance allowed for a 20 percent reduction in total structure thickness for arid conditions. It was shortly found that this provision was being abused and it was withdrawn. Later, following findings of the Field Moisture studies in the 1950's, the 20 percent thickness reduction was reintroduced with specific controls on defining arid conditions as being over 15 ft to water table and less than 15-in. annual rainfall. Current manuals still allow this reduction.

Standardized Test Procedures

As a result of the comprehensive study of the CBR test, a number of aspects of the test procedures were established or standardized. The following is a list of these determinations, most of which continue in use.

a. Dynamic (drop hammer) compaction should be used.

b. The 6-in.-diam (CBR) mold should be used.

c. Specimens should be not less than 4-1/2-in. in height.

d. The Modified AASHO or Proctor methods are to be used: 10 lb hammer, 18-in. drop, 2-in.-diam striking face, and five equal layers.

e. Specimens are to be soaked for 4 days with water available to both top and bottom (submerged) of specimen.

f. A satisfactory drainage time (before testing) is 15 min.

g. Swell should be determined during soaking and under specimen confinement by a weight representing the weight of overlying structure, but not less than 10 lb. Less than 3 percent swell was considered acceptable.

h. Test penetration should employ a 3-sq in. end area circular (1.95-in.-diam) piston.

i. A specimen should be confined by a weight representing the weight of overlying structure but of at least 10 lb during penetration.

j. Penetration should be at a rate of 0.05 in. per minute.

k. Load-penetration plots which are concave upward (penetration horizontal on plot) should be adjusted to have an initial straight section and the penetration scale should be shifted (to the right) to accommodate this adjustment (re-zeroed).

l. Determinations of CBR should be made for both 0.1- and 0.2-in. penetration.

m. Reference loadings are standardized at 1,000 psi for 0.1-in. penetration and 1,500 psi for 0.2-in. penetration.

n. The CBR at 0.1-in. penetration is used unless the CBR at 0.2-in. penetration is larger. Where the 0.2-in. CBR is larger, the test must be rerun. If the rerun confirms the larger 0.2-in. value, it is taken as the proper CBR.

CBR test determinations can be made on undisturbed samples from the field or in-place in the field as well as on remolded samples. Soaking is employed to represent the minimum strength expected during the life of a pavement. In some circumstances the existing strength or short-term future strength is needed. In these cases no soaking should be applied.

Selecting Design CBR

For design strength (CBR) determination, two methods were delineated as a result of the CBR test studies conducted in 1942-1945.

Method 1 - Conduct a modified AASHO compaction test to establish maximum density and optimum moisture content for the modified effort (55 blows per layer, 55,000 ft-lb per cu ft). Conduct CBR tests on three specimens, each at the optimum moisture content determined, but for three separate compaction efforts, i.e. 55, 25, and 10 blows per layer. From these, plot a curve of density versus CBR. Select the design CBR from this curve at a density of 95 percent of the maximum found from the modified AASHO compaction test.

Method 2 - Prepare specimens to define compaction curves for three efforts; 55, 25, and 10 blows per layer using the modified AASHO method. Conduct CBR tests on all specimens. Plot density versus molding moisture content, CBR versus molding moisture content, CBR versus density for constant values of molding water content. From the patterns of these plots and a range of water content dry of 55 blow effort optimum moisture content, and a minimum density of 95 percent of maximum for the 55 blow effort, select the design CBR.

Field CBR

Instructions for use of field in-place CBR apparatus as well as the equipment necessary were developed and reported as part of the comprehensive CBR test study (1942-1945). These continue to be much the same at present.

Swelling Soils

The CBR test studies also identified the need for special handling of swelling soils. The preferable practice of compaction near but dry of optimum moisture tends to maximize soil swell in emplaced soils. It was recognized that swell can be substantially reduced in potential by placing wet of optimum. This, however, results in an effective strength (CBR) much less than for the dry side compaction, and requires more protective overlying structure to support a particular loading.

Mold Size

Mold size was a concern in the CBR test studies (1942-1945) in reference particularly to materials having larger particles. It shortly became the practice to remove particles larger than 3/4 in. and replace them with an equal weight of material passing the 3/4 in. and retained on the No. 4 sieve. This was for 6-in.-diam mold specimens for compaction and CBR determination.

Stabilization of Soils

Stabilization of soils and soil-aggregates to improve performance in pavement structures was recognized as a functional option quite early. Test items were included in some of the early field tests (1940's), but primary dependence was on technology being developed by others. The initial manual guidance (early 1950's) was quite limited. The Corps undertook development of an improved manual which came out in 1955. This was still based on available technology, and much assistance was provided by the Portland Cement Association, the National Lime Association, and the Asphalt Institute. Cement, lime, and asphalt were the predominant stabilizing agents being employed as agents for upgrading soil and soil-aggregate materials to select material subbase, subbase, and base quality materials for use in pavements.

Early references to stabilization commonly intended to include blending aggregates or other superior soil materials into low quality soils to improve their behavior as pavement layers. Chemical agent stabilization was also included, and in some cases compaction was also considered to provide stabilization. More recently most references are intended to include only cementing type additives.

Many additives, other than cement, lime, and asphalt, have been considered or proposed as having the capability of stabilizing soils. Cognizance of the potential of a great many such additives was provided during the development of the 1955 manual and in following years by a substantial military

program concerned with expedient applications of soil stabilization. Military expedient concerns were focused more on upgrading very soft soils to something useable rather than conventional pavement applications aimed at improving soils to subbase or base quality, but the studies for one application provided guidance for the other.

While a number of these additives (other than cement, lime, or asphalt) have shown promise and a few have enjoyed substantial study and application for military purposes, virtually none have found application by the Corps for conventional pavement purposes. One exception, and one not Corps developed, is the lime-cement-flyash (LCF) found to serve well in applications at Newark Airport by the Port Authority of New York and New Jersey (about late 1960's). Corps doctrine now extends to use LCF stabilization.

The superior load distributing characteristics of stabilized layers, emphasized by elements of the AASHO Road Test Analyses (mid-1960's) were introduced into practice some 10 years later following substantial Corps studies. These studies resulted in the acceptance of layer equivalency factors in Corps criteria. The studies were discussed in greater detail in the previous section.

Unified Soil Classification System

Applicable and pertinent to all pavement work is the Unified Soil Classification System. In the late 1940's the Airfield Classification System developed by Casagrande was modified and adapted to United States geotechnical needs in a cooperative effort by the US Bureau of Reclamation, the Tennessee Valley Authority, and the Corps. This classification was soils oriented and not solely in support of pavement concerns. An established version of the Unified Soil Classification System was published by the Corps in 1953 including adaptations specifically for roads and airfields. In about 1960 the system was published as a Military Standard (MIL-STD 619).

The Unified Soil Classification System has found broad acceptance by the military and various other United States and international agencies (the Federal Highway Administration and until recently (1980's) the FAA were notable exceptions). Highway oriented personnel continued to use the AASHO (now AASHTO) Classification System but the FAA had its own classification system until changing to the USCS in the mid 1980's).

Field Moisture Investigation

On completing and reporting of the CBR test studies in July 1945 an extensive "Field Moisture Content Investigation" began. Main elements of this study involved the regular periodic examination of the moisture content beneath pavements of a number of in-service US Air Force airfields. The US Army Air Corps became the US Air Force in 1947.

It was broadly accepted, at the time, that the CBR rating for design should represent the minimum strength of materials in the pavement structure which it would obtain during its service life. This minimum strength was known to relate strongly to maximum moisture content in the prototype pavement layer materials, but what value this maximum might become in the structure was a matter of speculation.

The soaking of CBR specimens was adopted as practice to conservatively represent the maximum moisture content and minimum CBR for design purposes. It was not known how conservative the soaked CBR might be. The field moisture studies were undertaken in an effort to determine better the severity of moisture conditions to be expected in prototype pavements. Aspects of freezing, frost penetration, and thaw conditions were not a part of these studies.

The field moisture studies, extending from 1945 to 1963, included two studies of the potential for direct measurement of in-place moisture. This was accomplished by using moisture cells and some 11 years of seasonal sampling of the moisture content at various depths and cross-runway positions beneath in-service pavements. One moisture cell study conducted in 1945-1948 examined the potential for direct measurement of moisture using "Buoyoucos Blocks", porous plaster units. These cells were not satisfactory. The second moisture cell study conducted in 1955-1959 examined "Coleman" electrical (sheathed fiberglass) units. These also were not satisfactory.

The 11 year program of seasonal sampling of the moisture within in-service pavements permitted a number of significant conclusions or determinations.

a. Regardless of conditions induced during construction, the moisture in layers beneath the surfacing will attain a high percent saturation condition with small to no further variation. This condition will obtain in perhaps 3 to 5 years after construction.

b. Complete saturation, as for soils permanently below the ground water table, does not occur, but fine grained plastic materials will become 95 to

98 percent saturated. Materials of low plasticity will attain a condition a few percent lower. In extremely arid areas fine plastic materials may attain only 92 to 93 percent.

c. Moisture beneath pavements shows no relation to rainfall.

d. Seasonal variation in moisture is small to none.

e. Soaking CBR test specimens produces results about right to slightly conservative for subgrade soils. Soaking base type materials resulted in somewhat more, but not severely, conservative conditions.

f. Except for about 10 to 15 ft at runway edges, the moisture conditions showed no significant variation laterally beneath wide pavements.

A practice was adopted of allowing 20 percent thickness reduction from soaked CBR designs for arid areas having less than 15-in. rainfall and ground water permanently deeper than 15 ft. This practice continues to the present.

Standardized CBR for Base and Subbase

In the mid 1950's concern for questionable determinations of the proper CBR values for base materials and, to a degree similarly, for subbase materials led to revisions in the practices for strength design of bases and subbases. Because of the effects of processing samples for the laboratory CBR tests and because of the effects of the test mold, the need for test determination of CBR values on base course materials by testing was eliminated. Dependence for base quality became entirely dependent on the guide specification requirements with design CBR values assigned as follows:

Graded crushed aggregate	CBR-100
Water-bound macadam	CBR-100
Dry-bound macadam	CBR-100
Bituminous binder and surface courses, central plant, hot mix (employed as base)	CBR-100
Limerock	CBR-80
Mechanically stabilized aggregate	CBR-80*

* Tri-Service-Flexible Pavement Manual states: A blend of crushed and natural materials processed to provide a dense graded mix. LL = 25 maximum, PI = 5 maximum, and L. A. wear = 50 maximum.

Similarly for subbases, because processing and mold constraints were providing unconservative values, limiting values of CBR and constraints on plasticity and fines were introduced as follows:

Material	Maximum Design CBR	Maximum Permissible Value: Sieve in.	% passing sieve No. 10	% passing sieve No. 200	LL	PI
Subbase	50	3	50	15	25	5
Subbase	40	3	80	15	25	5
Subbase	30	3	100	15	25	5
Select material	20	3*		25*	35*	12*

* Suggested limits; not mandatory.

These materials continued to be subjected to CBR testing and were required to show test values as high or higher than used in design.

The limiting values introduced for subbases, and indeed many of those employed in base course guide specifications, are consensus judgement values based on results of the field moisture studies and on experience with the airfield condition survey and evaluation program conducted to provide design verification. It is worthy to note that conventional practice by others has tended to divide plastic and nonplastic behavior at a value of PI = 6. For Corps purposes, there was strong basis for a lower (PI = 5) value, and consultants regularly reviewing progress of these Corps developments gave strong endorsement of the lower value.

Strength Test and Material Characteristics
for Flexible Pavements

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CHAPTER 4

BITUMINOUS MIX DESIGN AND BEHAVIOR

Introduction

Following the early 1942 work leading to selection and development of the CBR methods for pavement design and the 1942-43 studies leading to CBR test development, it became necessary to initiate efforts toward mix design of bituminous surfacing materials. Asphalt mixes were being designed by following certain established rules and specifications. Designs employed were normally subject to adjustment by the construction engineer as his expert experience might indicate. Such procedure is satisfactory where engineers having long experience with local materials are available to supervise pavement construction.

Extensive Investigation of Mix Design

The exigencies of World War II brought a different situation, and methods for mix design were needed which could be followed by trained personnel who could use materials available in remote locations.

Work that came under the project title, "Investigation of the Design and Control of Asphalt Paving Mixtures" was initiated. The earliest work was "Comparative Laboratory Tests on Rock Asphalts and Hot Mix Asphaltic Concrete Surfacing Materials" which was reported 1 October 1943. These studies were done by the Tulsa District Office, Corps of Engineers.. A "Directive for Investigation of Stability of Asphalt Paving Mixtures" was dated 13 October 1943 and assigned to the Flexible Pavement Laboratory at WES. By April 1944, "A Proposed Project for the Field Investigation of Asphalt Pavements" was added to the investigation. Continuing funds for traffic tests, correlation studies, and final reports were added in 1946. A comprehensive report in three volumes was published in May 1948.

The Tulsa study compared four mix design methods that were used or developed at the time. These were the Hubbard-Field, Hveem stabilometer, Texas Punching Shear, and Skidmore test procedures. From these studies, the Hubbard-Field method was considered the most satisfactory. The method was also becoming widely recognized.

Marshall Stability Test

The Marshall apparatus had been developed by Bruce Marshall while with the Mississippi Highway Department. The Marshall equipment offered

capabilities equal to those of the Hubbard-Field. The Marshall equipment was light and portable for both laboratory and field use and adaptable to CBR testing equipment already adopted. The Hubbard-Field apparatus is large and heavy.

World War II concerns had strong influence on planning for development of airfield pavement design methods. Thus, consideration for the light weight fieldability of test equipment and adaptability to equipment already being used in the field were of prime importance.

The Marshall stability test used specimens 4 in. in diameter and (at least nominally) 2-1/2 in. thick. Prepared specimens were formed to this pattern in 4-in.-diam molds with compaction to the desired thickness. The test could be conducted on field cores (4-in. diam) cut from pavements in service or being constructed. Specimens deviating from the 2-1/2 in. thickness could be tested and the test values suitably adjusted.

Specimens are loaded diametrically (on edge). In concept this attempts to simulate the lateral thrust induced in a pavement immediately adjacent to a wheel load either directly from the load or as increased by acceleration, braking, or side thrust on turning.

The Marshall stability value was and still is the maximum load in pounds which a specimen could sustain while being failed in shear. The amount by which the (half-circle) breaking heads have come together at maximum load is the flow which is measured and recorded in hundredths of an inch.

Work undertaken at WES initially was to compare Marshall and Hubbard-Field apparatus and methods. This resulted in selection of the Marshall apparatus and initiation of a laboratory test program to develop test procedures and examine various aspects of gradation, filler, mix temperature, type aggregate, test, and field density for both sand asphalt and asphalt concrete. Further comparisons to Hubbard-Field methods were also included.

It was recognized that before laboratory tests on asphalt mixtures can be properly evaluated or the values obtained from such tests can serve as design criteria, the results of tests need to be correlated with behavior of pavements under actual traffic. Accordingly, from July to October 1944 the asphalt stability test section was constructed at WES. This included some 72 main items and 117 turnaround items each trafficked with three different loadings.

Correlation Tests

A third phase of the "Investigation of the Design and Cortrol of Asphalt Paving Mixtures" was the "Final Laboratory Correlation Tests." These correlations looked carefully at the Marshall equipment and compaction, at the test techniques, and at the materials.

Results of Bituminous Mix Studies

The investigation from method selection, through laboratory studies, the large field test section, and final correlation tests was an in-depth study lasting 5 years (1943-1948). Findings, developments, and established practices are listed in the following paragraphs grouped as general conclusions and specific conclusions.

a. General conclusions. The general conclusions were considered applicable for single-wheel loads ranging from 15,000 to 37,000 lb and dual-wheel loads up to 60,000 lb with tire pressures up to 100 psi. They are presented in their order of importance.

(1) The selection of proper asphalt content is the most important factor in design of an asphalt paving mixture.

(2) The Marshall method, as developed and presented, enables the proper asphalt content to be accurately selected.

(3) The criteria for selecting optimum asphalt and limiting values for satisfactory asphaltic concrete and sand asphalt are listed in the following tabulation.

Test Property	Specification Limit* Asphalt Concrete	Specification Limit* Sand Asphalt	Select Asphalt Content at Asphalt Concrete	Select Asphalt Content at Sand Asphalt
Stability	Min. 500	Min. 500	Peak of curve	Peak of curve
Flow	Max. 20	Max. 20	--	--
Unit weight total mix	--	--	Peak of curve	Peak of curve
Percent voids total mix	3-5	5-7	4	6
Percent voids filled with asphalt	75-85	65-75	80	70

* Stone filled sand asphalt would fall between the asphalt concrete and sand asphalt.

(4) Standard laboratory compactive effort for mix designs at 100 psi tire pressure is 50 blows to each face of the Marshall specimen using a 10-lb hammer falling 18 in. onto a 3-7/8 in. diam foot plate.

(5) The optimum asphalt content selected using the standard laboratory compaction is the same as would be found by full scale traffic tests for traffic of 500 to 1,500 coverages.

(6) For construction control, pavements should be compacted to 98 percent of the standard laboratory density.

(7) A high-quality base does not require a pavement to protect it from shear deformation, but for heavier wheel loads, thicker pavements tend to reduce consolidation in underlying materials.

(8) Bases of inferior quality may be protected from shear deformation by increased thicknesses of asphalt pavement of proper design.

(9) Thicknesses recommended as a minimum are as follows:

Base CBR	Minimum Thickness in Inches: 15,000 lb Wheel Load	Minimum Thickness in Inches: 37,000 Single or 60,000 Dual Load
40	4	5
50	3	5
60	2	4
80	2	3

(10) Higher stabilities are not as effective as additional thickness in preventing shear deformation.

(11) Recommended gradation limits are as follows:

Sieve Size	Gradation Limits for Percent Passing: Asphalt Concrete	Stone Filled Sand Asphalt	Sand Asphalt
3/4 in.	100	100	--
1/2 in.	76-100	92-100	--
No. 4	50-80	72-100	100
No. 10	35-60	55-82	75-100
No. 20	22-49	40-66	50-82
No. 40	12-38	28-52	35-65
No. 80	7-26	16-36	18-44
No. 200	3-12	5-16	8-20

(12) Inclusion of a No. 20 sieve is desirable for control of aggregate.

(13) Recommended gradation limits for filler* are as follows:

Grain Size	Limits Percent Passing
Passing No. 200	100
0.05 mm	70-100
0.02 mm	35-65
0.005 mm	10-22

Filler should be well graded from coarse to fine.

(14) Filler is a void filling material which increases both stability and density. Maximum amounts of filler should be 20 percent for sand asphalt and 12 percent for asphalt concrete.

(15) The use of the most asphalt (also void filling) and least filler consistent with limiting test property criteria is considered desirable.

b. Specific conclusions. Specific conclusions cited as a result of the extensive mix design investigation are listed here in no particular order of preference:

(1) Marshall apparatus.

(a) Marshall equipment is an excellent tool for design and construction control of asphalt paving mixtures.

(b) Test results using Marshall apparatus compare favorably with those obtained using Hubbard-Field equipment.

(2) Stability.

(a) The peak of the stability versus asphalt content curve is an excellent criterion in selection of optimum asphalt.

(b) The stability value by itself is not a satisfactory indicator of ability to support traffic.

(c) No benefit is obtained from higher than minimum required stability for pavement on CBR 80 or higher base.

(d) Equal stability between mixes does not assure equal performance.

(3) Flow.

(a) The flow, as a part of the stability test, indicates relative plasticity of an asphalt mixture.

(b) Increase in asphalt content increases flow value.

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* Minus 200 material used in mix.

(c) Generally flow values above 20 indicate a mix that will displace under traffic regardless of the stability.

(d) Stabilities of two mixes can be compared only if their flow values are satisfactory.

(4) Unit weight total mix.

(a) An increase in asphalt content increases unit weight to some maximum beyond which unit weight decreases.

(b) In general an increase in unit weight produces an increase in stability.

(c) Density or unit weight in pounds per cubic foot cannot be used to compare two unlike mix designs.

(5) Unit weight aggregate only.

(a) The amount of asphalt required to produce maximum density of a mixture considering the aggregate only is less than that required to produce maximum total weight.

(b) Peak of the aggregate only curve indicates that asphalt contents leaner than field behavior is desirable; so peak of the curve is not a criterion for selecting optimum asphalt for mix design.

(6) Percent voids aggregate only. The percent voids in the aggregate only is not a significant factor in behavior of pavements under traffic and is not adopted as selection criteria.

(7) Percent void total mix. The percent voids in the total mix is a useful index for evaluating pavement quality.

(8) Percent voids filled with asphalt.

(a) The percent voids filled is a useful index for evaluating pavement quality.

(b) Very harsh or open-graded aggregates have lower percent voids filled than well-graded mixes.

(9) Effect of wheel load.

(a) The density of all pavement items was increased by traffic and resulted in change in all test property values.

(b) The change in test property values was the same at equal coverages for all loads.

(c) The 37,000 lb single and 60,000 lb dual-wheel loads had about equal effects on base and subgrade, but both were more severe than the 15,000 single load.

(10) Compactive effort.

(a) Test property relationships developed in field construction are similar to those obtained in the laboratory. Density of pavement increased as passes of the construction roller increased.

(b) Increase of density by traffic compaction reduces the quantity of asphalt applicable to an established percent of voids filled. It is necessary to design pavement mix for an asphalt content satisfactory after traffic to prevent the mix from becoming plastic due to density increase.

(c) The compactive effort applied by traffic was a function of repetitions and tire pressure and did not relate to wheel load.

(d) Results of these tests, between 500 and 1,500 coverages, are applicable for determining laboratory compactive effort to be used.

(e) Increase in laboratory compaction increases both stability and density.

(11) Effect of aggregate type.

(a) Pavements on high-quality base showed little variation with aggregate type for properly proportioned mixes.

(b) On low-quality bases asphalt concrete made with limestone aggregate was more effective in preventing shear deformation than sand asphalt, but this type relation was not fully studied.

(c) Laboratory test properties were little affected by type of coarse aggregate when less than 35 to 40 percent coarse aggregate was used in the mix.

(d) When more than about 35 to 40 percent coarse aggregate was used in the mix, the type of aggregate was very significant. Slag was highest, crushed limestone or gravel intermediate, and uncrushed gravel lowest in stability.

(12) Effect of gradation.

(a) Test properties of sand asphalt are improved with additions of coarse (No. 10 to No. 40) sand up to about 50 to 60 percent of the total.

(b) Test properties of asphaltic concrete are improved with addition of coarse aggregate up to about 50 percent and are satisfactory to about 65 percent of the total.

(c) Increase in maximum size of aggregate from 1/2 to 1 in. improved test properties.

(d) For the limited comparisons made, gap-graded mixtures showed test properties equivalent to those for well-graded mixtures having the same quantity of coarse aggregate.

(e) Traffic testing of well-graded and poorly graded mixtures did not reveal any important differences.

(13) Filler.

(a) The use of good quality filler will improve test properties and reduce asphalt required, but excessive filler is detrimental from a durability standpoint.

(b) A maximum of 20 percent limestone dust or portland cement but only 15 percent of other type filler improved test properties for sand asphalt.

(c) The maximum for filler in asphaltic concrete is a function of the sand content.

(d) Limestone dust and portland cement performed satisfactorily as fillers; loess, clay, and clay-loess were inferior fillers; and the -200 sand tested was not satisfactory.

(14) Effect of mixing temperature. Aggregate heating temperature between 300 and 400°F prior to specimen preparation had no appreciable effect on any test property except stability which increased as the temperature increased.

(15) Effect of penetration grade of asphalt cement. Penetration grades between 50 and 135 had no effect on any measured test property except stability which increased with decrease in the penetration grade.

(16) Test techniques.

(a) Apparent specific gravity of aggregates by ASTM methods was tentatively recommended for use in computing properties of bituminous mixtures.

(b) No change in stability resulted from holding specimens in the 140°F water bath longer than 10 minutes.

(c) Test specimens may be placed in the water bath immediately after molding and tested after 20 minutes in the bath with no detrimental effect on test properties.

(d) No significant change in test properties resulted from curing specimens in air for up to 350 days before testing.

(e) The Marshall equipment flow meter gave satisfactory values of flow in the field testing apparatus.

(f) The relation between stability and rate of deformation follows a definite pattern for which stability can be predicted for minor changes in rate of deformation. The relative quality of mixes did not vary with changes in rate of deformation.

(g) The standard compaction mold used in the Marshall test is satisfactory for laboratory tests providing no particles larger than 1 in. are involved.

Adopted Practice

A June 1948 report, "Formulas and Procedures for the Design and Control of Asphalt Paving Mixtures" summarizes the adopted practice following completion and reporting of the extensive investigation of mix design and adoption of Marshall stability methods. Procedures for mix design for 100 psi tire pressure pavements (load magnitude had been found to have virtually no effect on surfacing performance) are included and are much the same as presently practiced.

Fuel Spillage/Bitumen versus Asphalt

All early work used asphalt as the mix binder, but the United States practice of referring to bituminous as including both asphalt and tar was already in use. The emphasis on tar in relation to resisting spilled solvents strengthened the use of the term "bitumen" to include both asphalt and tar. British practice does not follow this usage and can be misleading. This is further aggravated by United States practice influencing some British (and other countries) pavement practitioners.

Initial Spillage Concerns

As early as February 1946 a study was initiated on the detrimental effects of solvent spillage. These included gasoline, kerosene, lubricating oil, and hydraulic fluid. The study reported in December 1947 concluded that bituminous pavements can be protected from detrimental effects of solvent spillage by use of suitable admixtures or, as a temporary expedient, by protective treatments applied to existing pavements. Pavements using coal tar as binder for the mix was considered most satisfactory. For surface treatments, a tar seal was considered satisfactory. The commercial products vermiculite and solac were satisfactory but more expensive than tar. The jet fuel spillage and blast problems were still to come.

High Pressure Tire Studies

After the end of World War II, following the B-29 came the B-36 aircraft, with tire pressures exceeding the 100 psi range and approaching 200 psi. Tests planned in late 1948 and carried out in 1949 were reported in May 1950 as "Investigation of Effects of Traffic with High Pressure Tires on Asphalt Pavements."

The existing asphalt stability test section (see 10!D 3-254) was subjected to traffic of 30,000 lb and 200 psi tire loads in single and in a twin-tandem (120,000 lb) configuration. It was concluded that:

a. Asphalt paving mixtures can be designed which will be satisfactory under traffic of tires inflated to 200 psi.

b. Traffic with 200 psi tires results in densities 2 to 2-1/2 lb per cu ft higher than traffic with 100 psi tires.

c. Pavements showed a marked effect of asphalt content with lower asphalt content pavements showing superior behavior.

d. The higher tire pressure mix design problem appeared to be one of adjusting lower pressure procedures to give lower design asphalt contents consistent with the higher densities from the 200 psi traffic.

e. On the high quality subgrade, pavements using uncrushed gravel for coarse aggregate showed equal performance to those using crushed limestone.

f. Rutting was more severe in the thicker than in the thinner pavements.

g. The twin-tandem gear at 120,000 lb load was no more severe on similar mix designs than the 30,000 lb single-wheel load where all tires were at 200 psi.

While the tests indicated a mix-design method for pavements to support 200 psi tires could be developed, the results from the tests were not sufficient for that purpose.

In September 1949 a special investigation was undertaken for the Navy. The Navy's F9F aircraft had only an 8,000 lb wheel load but had tires inflated to 240 psi. It was decided that some of the thinner structural sections of the existing asphalt stability test section (see F 3-254) would accommodate the 8,000 lb wheel load, and surfacing variables would provide valuable knowledge on response to the 240 psi tire loads. These tests were completed by November 1949 and reported in June 1950 as "Effects of Traffic with Small High-Pressure Tires on Asphalt Pavements."

From these tests the following conclusions were presented:

a. Traffic of the small 240-psi tires with an 8,000 lb wheel load was markedly more detrimental to the pavements than the larger low pressure tires carrying a 15,000 lb wheel load used in the 1944 testing.

b. Satisfactory asphaltic pavements can be designed for small 240-psi tires.

c. Satisfactory design appears to be a problem of selecting proper asphalt content and using well-graded aggregate to assure stability.

d. Data obtained were considered sufficient to establish tentative criteria for design of asphaltic concrete wearing courses but not for sand asphalt mix design.

e. Tentative criteria suggested are listed in the following tabulation.

Test Property	Values
Stability	1,000 lb minimum
Flow	16 maximum
Percent voids	4 to 6
Percent voids filled with asphalt	75 to 82

f. Current laboratory compaction will not produce densities comparable to those produced by small 240-psi tire traffic.

g. It was tentatively proposed that Marshall test specimens be compacted with 75 blows on each face (10-lb hammer, 18-in. drop, 3-7/8 in. diam footplate) and the following criteria used for selection of optimum asphalt content:

Test Property	Selected Asphalt Content At
Stability	Peak
Unit weight total mix	Peak
Unit weight aggregate only	Peak
Percent voids	5
Percent voids filled with asphalt	78

h. Effect of parked loads and locked wheel turns were not as severe as slowly moving loads.

A small test section was built and traffic tested, which confirmed the tentative criteria (e and g above). These were the effective criteria with the exceptions of jet blast and fuel spillage for the early to mid 1950's for 200-psi tire pressure.

Jet Blast and Fuel Spillage

Increasing use of jet aircraft (1951-1952) brought the problems of blast effects on pavements and of jet-fuel spillage softening and leaching asphalt binder. Spillage and blast in combination was especially severe. Laboratory and fieldwork indicated that low voids asphalt pavements were more resistant to spillage of jet fuel. This led to a change in the 200-psi tire pressure voids criteria from 4-6 to 3-5. It also led to a tightening of the gradation requirements for surface course mixtures. It was also learned that rubber-tired rolling would help to seal the surface against fuel intrusion. Surface fuel would evaporate, but fuel penetrating pores or cracks were particularly effective in softening the asphalt.

Concern for jet blast effects on asphalt pavements began in 1951 with a study of heat and blast effects of jet aircraft. An interim report was issued in July 1952. Concern for fuel spillage and combined spillage and blast followed immediately. Jet-blast and fuel-spillage tests were conducted at Hunter AFB and reported in March 1952.

Tar Rubber

Since tar is not soluble in petroleum products, the potential for use of tar as the binder for bituminous pavements was immediately apparent. Tar, however, has a lower softening temperature than asphalt. It had been found that this problem might be ameliorated by blending rubber with tar. A laboratory investigation of use of tar and tar-rubber blends for pavement mix binders to resist spillage and blast was undertaken and reported late in 1952.

The program of study for heat and blast and for tar rubber pavements and tar sealers became quite extensive. Full-scale tar-rubber test sections were constructed and tested (1953-1957). Further heat and blast studies were completed (1954). Tar-rubber pavements were constructed. There are many literature references to aspects of this program; covering the period up to 1974 when a tar-rubber overlay report was issued. Not all references are included in the bibliography, but a few selected references are cited.

Except for one or two early jet aircraft (B-45 and F-80), jet blast was not a problem for any aircraft. Unduly severe operations, however, on even high-quality asphaltic concrete pavements were a problem. Aircraft having downward blast impingement angles and small above-pavement heights of the jet stream were marginal for causing surface erosion when operating at full power in fixed positions for extended periods. For any aircraft, stopped or

tethered, use of afterburners such that the blast impinges in any one area for even a few seconds can cause distress. The afterburners caused no problem while the aircraft is rolling forward.

Tar Rubber Pavements

Tar-rubber pavements had blast resistance comparable to that of asphalt concrete and were not softened by fuel spillage. They did, however, tend to crack due to shrinkage, and if the cracks were not carefully maintained (kept sealed), fuel would enter and soften underlying asphalt layers. Fuel entrapped in and below cracked pavements would not cure out readily. Work with RT-12 tars and other special tars has helped the cracking problem, but a complete solution has not been found.

Both tar and tar-rubber mixes were satisfactorily designed and placed using the methods developed for asphalt mixes, except that the temperatures were changed to accommodate the particular binder used.

Tar Seals

Tar seals, some commercial products, and some selected liquid tar forms were studied for application to asphalt surfaces to protect them from fuel spillage. Tar seals are effective for protection against spillage as long as they can be maintained against leakage. General experience has shown that virtually all such seals are subject to cracking or other leakage. When fuel gets beneath a seal, it is more effective in softening an asphalt than fuel directly on the surface of an unsealed asphalt.

Shell Aggregate

Beginning in 1951 and following a report of studies undertaken in 1954 there developed an interest in the use of shell aggregate for hot-mix asphalt. It was found that, while shell aggregate would not meet particle shape criteria, blends of sand, shell, and screening could be used to make mixes that meet 200 psi design criteria (1,000 minimum stability was still the criteria in effect). These mixes would satisfactorily carry 200 psi traffic (1,500 coverages). Spillage problems for shell asphalt mixes were about the same as for conventional mixes. Blast resistance of shell aggregate mixes was never studied.

Porous Aggregates

In 1952 it became apparent that the established mix design methods using the ASTM apparent specific gravity were not satisfactory for porous (above 2.5 percent absorption) aggregates. Work by the South Atlantic Division and

by WES in 1952 resulted in development of the bulk-impregnated specific gravity test. Further studies by WES, reported in August 1953, developed the following proposed criteria for surface course mixes to support 200-psi tire traffic and using bulk-impregnated specific gravity.

Test Property	Proposed Criteria
Percent voids total mix	1.5-3.5
Percent voids filled	80-90

Other mix design criteria for 75 blow Marshall remained the same. The minimum stability was 1,000.

Firm criteria have since been introduced which specify the use of bulk-impregnated specific gravity for mix designs with aggregates having absorption greater than 2.5 percent. Percent voids total mix are consistently 1 percent lower and percent voids filled 5 percent higher than for mixes with nonporous aggregates designed using ASTM apparent specific gravity.

Emulsified Asphalt

Emulsified asphalt came into more general use in the early 1950's. The Corps made a limited study of using emulsified asphalt in hot-mix asphaltic concrete. It was found that emulsified asphalt can be used to make high quality hot mix and that the standard Corps procedures could be used. A higher aggregate temperature of 475°F (used to drive off the water) appeared to be necessary.

Channelized Traffic

Channelized traffic which occurred with the introduction of the B-47 and the B-52 aircraft brought some pavement distress problems in the early 1950's. Study of pavement behavior under channelized traffic included concern for the pavement mix design. It was found that no distress was directly attributable to the asphaltic concrete surface or binder courses. There was indication that higher densities and lower voids total mix would be a future concern. These studies included 23 Air Force bases and were made between 1954 and 1958.

Effects of Heavy Loads

By the mid 1950's there was increased concern for heavy loadings and tire pressures to 300 psi and above. The B-52 aircraft had enjoyed some growth in gross weight, and a heavier follow-on bomber was under development (the WS-110 which became the XB-70). Top level Air Force officials were endorsing the use of rigid pavements for heavy-duty airfields even at some unfavorable cost

differential. Their claim was that flexible pavements could not satisfy Air Force requirements. Pilot tests were conducted by WES in 1956 which used twin-tandem loads up to 325,000 lb and tire pressures up to 325 psi. The objective was to provide information on the effects of amount and type of asphalt and aggregate gradation on performance for these intensive loadings. It was thought such loads would require a reduction in asphalt content from laboratory optimum as determined by Marshall procedure (75 blow and stability minimum still 1,000) or a paving mixture with great resistance to densification so that required voids (to avoid flushing) would be retained in the pavement under traffic. In these tests all traffic was applied while pavement temperatures were above 90°F, since it was recognized that at lower temperatures asphalt mixes are much less subject to change (more stable and less tendency to densify) under traffic.

Proof Tests

Before these 1956 tests were finally analyzed and reported in 1962, both the Kelly AFB (June 1957) and the Columbus AFB (December 1959) tests had been conducted and had some influence on the 1962 reporting of heavy-load effects on mix-design requirements. The proof tests were mandated by Congress in response to reactions from industry resulting from the Air Force's stated preference for using rigid pavement on heavy-duty airfields. The tests were primarily structural, but had significant impact on mix design of flexible pavements.

An Air Force policy was instituted (1953) which required use of portland cement concrete (rigid) in certain critical areas of airfield pavements and permitted a 5 percent cost premium in favor of rigid pavement in other areas. Apparently asphalt industry concerns led, in February 1954, to hearings by the Subcommittee for Special Investigations of the House of Representatives Committee on Armed Services. At that time there was no demonstrated difference in either structural capabilities or in maintenance requirements between the two basic (rigid and flexible) pavement types. There was however, as a result of both investigational tests and performance observations, a demonstrated difference in behavior with regard to fuel spillage and jet blast. In its May 1954 report the subcommittee concurred in the Air Force classification of all aprons and the 1,000-ft ends of runways as critical areas, and in the requirement that portland cement concrete pavements be used exclusively for these areas. The subcommittee did not find justification for a 5 percent premium in

favor of rigid pavement for other areas. In August 1954 the Department of the Air Force placed these concepts in its criteria for pavement type selection. At the time, the Air Force restated a preference for rigid pavement but admitted to a lack of confirming evidence.

Porpoising Effects

By late 1955 the channelized traffic and bicycle gear porpoising problems had been encountered. In December 1955 the Air Force changed criteria to specify that all pavements on which aircraft are normally operated, parked, serviced, or maintained should be classed as primary-use pavements and that all primary-use payment should be constructed of PCC. A proof-test program was undertaken to assess the design criteria needed to accommodate channelized traffic, both flexible and rigid pavement, and to determine the ability of contractors on a typical Corps project to meet the extended requirements.

Proof Tests, Kelly and Columbus AFB

Testing was arranged at Kelly AFB, Texas. The tests were primarily structural, but results impacted on asphalt mix design. The asphalt content selected for the tests was somewhat higher than developing technology indicated as satisfactory, but it was decided not to attempt this late an adjustment of criteria in the test comparisons. No mix problems were anticipated. All traffic was applied to represent hot-weather (above 90°F temperature) conditions.

Some shallow rutting of the flexible pavements resulted. Asphalt industry representatives contended that the application of only hot weather traffic in an amount to represent the entire life of the pavement was an unduly severe requirement. Further congressional subcommittee hearings were held resulting in suggested additional testing. This led to the tests at Columbus AFB in Mississippi in 1957 and 1958. Mix designs (now 1,800 stability) were used which were expected to accommodate the higher tire pressures, increased repetitions, and the warmer climatic region involved. These tests indicated that flexible pavements, including mix design, could provide for support of B-52 aircraft in other than critical areas. It was allowed that concrete keels in runway interiors would provide good insurance.

It may be interesting to note here that the Kelly AFB pavements in actual service since the proof tests have given good service, without wheel path depressions, for their anticipated functional life.

Results of Heavy-Load Tests

The 1962 reporting of heavy load preliminary tests to study pavement mix design for very heavy gear loads produced several notable findings and conclusions which are listed below.

a. The initial degree of compaction attained in construction of the various test paving mixtures ranged from about 98 to 102 percent of the laboratory Marshall density.

b. Traffic increased density beyond as-constructed density in every case.

c. All dense-graded paving mixtures made using standard paving grades of asphalt flushed when the voids total mix dropped below about 3 percent.

d. The Corps standard dense-graded mixtures performed better under traffic and showed greater resistance to densification than other gradings studied.

e. For traffic tests simulating B-52 loading on standard dense-graded paving mixtures using a standard paving grade of asphalt (85-100 penetration), the indicated optimum asphalt content was about 20 percent lower than that determined by the standard Marshall design procedure.

Recommendations from this 1962 reporting of the 1956 heavy load testing included use of Marshall optimum minus 20 percent for otherwise standard procedures for climatic areas comparable to that of Vicksburg, Mississippi where the tests were conducted. For other areas, the design asphalt content was to be varied in accordance with a 1957 study and criteria in engineer manuals. More extensive tests were to follow these pilot tests, but a shift in emphasis from bombers to missiles led to reduced program funding, and no further tests were undertaken.

Temperature Index

As a result of the WES heavy load tests and the Kelly AFB tests, a 20 percent reduction in asphalt content was recommended for pavements for channelized (30,000 coverages) traffic. The same tests, however, indicated that the compacting effect of traffic varies with pavement temperature so that the 20 percent reduction is not likely applicable in all climates. A 1957 study devised a temperature index and recommended the following adjustments to Marshall optimum asphalt determinations.

Bituminous Pavement Temperature Index	Asphalt Penetration Grade	Bitumen Content: Channelized Traffic	Bitumen Content: Nonchannelized Traffic
Negative	120-150	Optimum	Optimum +10 percent
0-40	100-120	Optimum	Optimum
40-100	85-100	Optimum -10 percent	Optimum
101 and above	60-70	Optimum -20 percent	Optimum -10 percent

The temperature index is a summation of the increments of average monthly maximum ambient temperature above 75°F. When the average monthly maximum for the warmest month is below 75°F, the difference is taken as the negative value of temperature index. Attempts to base the climatic variation pattern on the already established degree-days were not satisfactory.

At a conference in May 1968 the values of temperature index used as climate divisions (40 and 100 in the above) were adjusted downward to 20 and 80 and are used for guides in selection of asphalt penetration grade in "Bituminous Pavements" Standard Practice Manual (RADING 5-822-8) dated December 1971. The current manual (dated July 1987) has further adjustments.

<u>Gyratory Tests</u>

The 50 blow Marshall method for 100 psi pavements and 75 blow Marshall method for early 200 psi pavements yielded compaction densities sufficiently close to that experienced in pavements in service to provide a basis for satisfactory mix design. Additional increases in tire pressure and increased repetitions due to channelized traffic indicated the need for further changes. Simply increasing the blows on Marshall specimens did not provide a solution. Also, the problems with porous aggregates and the variety of specific gravity test methods, none of which served perfectly, seemed to indicate a need for specimen preparation that would better represent the construction rolling and effects of traffic wheel loads. This led, from initial concepts as early as 1954, through equipment developments of 1956 to 1959 and test methods reported in February 1962 to the gyratory testing equipment and mix design methods.

Gyratory methods permit design of mixes for very high tire pressures and high levels of load repetition. In later manuals, guidance required use of the gyratory for mix designs for pavements to support over 250-psi tire pressure loadings or channelized traffic.

A 1,800 Stability Value

In the late 1950's, adjustments to mix design for the higher field densities being produced by channelized traffic and higher tire pressures included raising the minimum stability value from 1,000 to 1,800.

Epoxy Asphalt

In the late 1950's Shell Oil Company developed an epoxy asphalt. Epoxy-asphalt concrete, although several times the cost of asphalt concrete, had superior tensile strength. It appeared to have potential for use in thin layers (about 3/4 in.) directly on a quality base, thereby substituting for several inches of asphaltic concrete. The epoxy-asphalt concrete was also resistant to fuel spillage and blast.

The fuel spillage and blast resistant character of epoxy-asphalt mixes were of particular interest, and trial installations were made at Air Force bases and some civil airfields for protection of existing asphalt pavements from fuel spillage or blast. There were also some trials on portland cement pavements for correction of surface deterioration or FOD problems by use of only a thin treatment in areas not subject to fuel spillage and blast effects.

By 1963 the Corps had a guide specification for epoxy-asphalt concrete and in 1963 some 20 airfields which had test or trial installations in place and in use were inspected. A report of these tests were published in early 1965. The following conclusions were reported.

a. Epoxy-asphalt pavements are sufficiently resistant to be used in areas subject to fuel spillage and blast from jet-type aircraft.

b. Thin overlays are subject to cracking, but the cracks do not spall or ravel.

c. Climate has a direct effect such that the colder the climate the more tendency to cracking.

d. If the random shrinkage cracking can be controlled, epoxy-asphalt pavement will be a very good maintenance material for both portland cement and bituminous pavements.

The cracking problem substantially weakened the potential of epoxyasphalt concrete. Shell Oil Company tried several alternate mixtures to improve resistance to cracking and interest continued for a few years, but early prospects were never realized and use has diminished.

Slurry Seals

In the early to mid 1960's slurry seals came into use and popularity. The Corps followed developments and field applications and performed laboratory and limited field tests of its own until the early 1970's.

In 1969 the Corps adopted (with permission) the Wet Track Abrasion Test from Chevron Asphalt Company with some slight modification. The Corps issued the Instruction Report S-69-1 in March 1969 explaining the use of the WTAT for developing a (starting) job mix formula for a slurry-seal mixture. Basic purpose of the WTAT is to measure wearing qualities of thin, fine aggregate bituminous surfacings such as slurry seals under wet conditions.

By 1971 a guide specification was in effect (CE-807.23) and an appendix in the "Bituminous Pavements Standard Practice" manual (5-822-8) presented guidance on design, proper use, and application of slurry seals. A June 1975 report (IR S-75-1) presents guidance for facilities engineers in selecting, designing, and applying slurry seals.

Slurry seals can fill narrow, nonworking cracks. They can improve skid resistance and appearance of pavements. Tar-emulsion slurry seals can provide resistance to fuel spillage, especially if a two-layer seal is applied. There is a wide difference in behavior of slurry seals between those well designed and constructed and those not so carefully done. Aggregates must be harsh crushed material. Gradations need to be carefully controlled and chosen for the conditions treated. Curing must be adequate, and cured applications need to be rubber-tired rolled for firm seating and aggregate interlock. Even with proper design and emplacement, slurry seals are not recommended for intense traffic support.

Aggregate Blending

Blending of aggregates of various size-ranges or from more than one stock source to provide gradations to satisfy specified gradation limits had always been a trial and error process. Substantial knowledge, skill, and good fortune was required to gain a satisfactory job gradation. With the advent of computer capability and suitable mathematical developments, it became possible to devise a computer method for aggregate blending. In 1970 the Corps issued an Instruction Report S-70-5 for that purpose.

Porous Friction Course

In the late 1960's responding to concern over hydroplaning the British developed an open-graded surface mix and made installations on a number of

NATO airfields in England. This is about the same time that grooving of pavements (including bituminous pavements) came into practice. Along with or closely behind the British a number of United States agencies studied and developed open-graded surfacings. USAF Civil Engineers also undertook open-mix study and developments. During 1971 to 1973 WES studied and developed a porous friction course (PFC) for the Corps.

Porous friction course is variously referred to as open-graded surface, popcorn mix, plant-mix seal, and others. These 1971-1973 studies were the basis for Corps guidance and, cooperatively, for the FAA as well. A February 1975 reporting for the FAA includes the following guidance.

a. Use PFC only on structurally sound pavement.

b. Both too soft and too hard asphalts have adverse aspects, so 85-100 penetration grade asphalt is recommended.

c. A maximum LA abrasion loss of 25 and sodium or magnesium soundness percent losses of 9 and 12, respectively, are recommended.

d. Minimum thickness of PFC should be a minimum of 3/4 in. and a maximum of 1 in.

e. A suggested design procedure for PFC mixes would be the California Division of Highways Centrifuge Kerosene Equivalent test (CKE, method 303-E) including the surface area K factor, viscosity temperature relations to establish mixing temperatures, and permeability tests to verify desired porosity.

f. A PFC guide specification was compiled and recommended for use.

Vibratory Compaction

By the mid 1970's vibratory compaction equipment had become widely available and was being applied to compaction of bituminous layers. The Corps undertook a field test study at WES to evaluate the effectiveness of vibratory rollers in compaction of hot-mix asphaltic concrete and rubberized-tar concrete. It had become necessary, to satisfy Air Force needs, to develop near 100 percent of 75 blow Marshall density during initial construction of heavy duty pavements. This was demanding on contractors, particularly highway contractors not equipped for heavy pneumatic roller compaction.

The studies undertaken were reported in June 1976. They established that vibratory (of the type used in testing, Buffalo-Bomag BW210-A, and Dynapac CC-50A) is satisfactory for compaction of high quality bituminous concrete pavements. Properly employed, they can provide densities meeting Air Force and Corps requirements.

Viscosity Grading

During the later 1970's and the early 1980's viscosity grading of asphalt came into common use to replace the earlier established system of penetration grades. Nominal equivalents were as follows:

Penetration Grade	Viscosity Grade
40–50	AC–40
60–70	AC–20
85–100	AC–10
120–150	AC–5
200–300	AC–2.5

There has been a parallel concern for asphalts graded at 77°F not having the same relative behavior at higher (275°F) temperatures. This has led to the use of Pen–Vis numbers for asphalt cements. These are illustrated in "Bituminous Pavements Standard Practice" manual (5–822–8/AFM 88–6, Chap. 9, July 1987).

Bituminous Mix Design and Behavior

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CHAPTER 5

RIGID PAVEMENT DESIGN METHODOLOGY

Introduction

Shortly after the Corps was assigned responsibility for the design and construction of military airfields in November 1940, two major problems required immediate attention. The first problem was the advent of heavy bomber aircraft represented by the B-17 Flying Fortress and the B-24 Liberator. These aircraft had just gone into production and were of a design to produce single-wheel main gear loadings of 35,000-lb with a maximum gross aircraft weight of 75,000-lb. This loading was three to five times greater than any highway or airfield loading designers had dealt with previously. The second problem was a lack of rational or valid design method by which either rigid- or flexible-type pavement could be designed to carry these loadings.

A simple, practical and uniform method of design for use in all parts of the world was required. In 1941 after a thorough study of established design procedures, nearly all for highways, the Corps embarked upon a pavement investigational program to obviate the use of untried methods; ensure adequately designed pavements; provide methods not subject to variation occasioned by arbitrary cost differentials of local competitive materials; avoid reductions in pavement thickness in order to balance cost; and establish procedures that would readily lend themselves to further development through tests, investigations, and study of actual pavement behavior.

The Corps' pavement investigational program encompassed all aspects of pavement design and behavior, but this section will be concerned with structural aspects of rigid pavements.

Westergaard Design Method

From a variety of design practices and methods in use for rigid pavement design, the procedures by Westergaard showed the greatest promise to serve the Corps' needs. H. M. Westergaard had published a theoretical rigid pavement design method for the Bureau of Public Roads in 1926 (Circular loaded area, center-slab loading, liquid subgrade). He made some adjustments and gained some verification from the Arlington Road Tests, and (1939) he extended the procedure to "Stresses in Concrete Runways of Airports."

a. Verification tests. In 1941 the Corps undertook a task to check the validity of Westergaard's center slab loading theory. The "Wright Field Slab Tests" reported in 1942 and 1943 placed static and dynamic loads by dropping a loaded aircraft tire on 20- by 20-ft concrete slabs. Principal conclusions were that the Westergaard equation accurately predicted the critical loading at which structural failure occurred under a center slab loading condition, and that dynamic loadings produce no greater stresses in a concrete slab than static loadings of equal magnitude.

b. Dynamic loading. With this encouragement it was next necessary to learn how dynamic loadings from landing aircraft would relate to the static loadings of standing aircraft. It was common practice for bridge design to consider an impact factor added to the static load to account for the spring-bounce of vehicles, and these vehicles did not leave the ground to impact on returning. In late 1941 flight tests and taxi tests with a B-26 Medium Bomber at Dayton Municipal Airport, Ohio, showed that for normal operations an aircraft at touchdown remains partly airborne so that pavement loads are less than static. A study of in-service pavements showed earliest signs of pavement distress to occur in areas of taxi traffic on taxiways, apron taxilanes, and runway ends. This confirmed that the static load could be taken as critical for design.

c. Center slab loading. The tests also carried implications that the Westergaard center slab loading is not the most critical, and that repetitive traffic has significance. All early thinking involved comparison of some maximum load magnitude to a limiting pavement support capacity.

Traffic tests in 1942 and 1943 at a number of existing airfields further strengthened a conclusion that the center slab loading theory, without modification or a suitable safety factor, was not sufficiently conservative for design purposes. By 1943 there was a definite need for rigid airfield pavement design criteria for Corps use. Accordingly, the Corps published a criteria based on the Westergaard analysis for interior slab loading, but the criteria contained a conservative safety factor and required load transfer or thickened edges at all construction and expansion joints.

Lockbourne Traffic Tests

The mounting weight of aircraft and early findings indicating that existing designs were truly optimistic strongly supported the need for more comprehensive studies. The 80,000-lb B-17 and B-24 bombers were in active

service, and the B-29, forecast to weigh 120,000-lb, was being developed. Accordingly, in June 1943 the first full-scale accelerated traffic tests under completely controlled conditions was undertaken at Lockbourne Army Airfield, Ohio.

a. Repetitions of traffic. It had become recognized that repetitions of load, in addition to magnitude, are of significance, and studies (see the section on traffic and loading) had established that 5,000 coverages were representative of a 10-year plus pavement life. This was needed for programming traffic on the "Lockbourne No. 1 Test Track."

b. Edge loading. For comparative analysis, Westergaard had developed edge load equations. Formal publication was in the Transactions of the "American Society for Civil Engineers" in 1947 and 1948.

c. Results of traffic tests. Basic conclusions developed from the Lockbourne No. 1 Test Track studies are listed below.

(1) Stresses produced in a pavement slab by either traffic or static loadings are more severe when the loading is applied at the corners and edges of a slab than when applied at the center.

(2) The Westergaard edge load equations developed in 1943 were valid for a single loading condition, but an additional design factor had to be applied to properly account for stress repetitions (fatigue), temperature, gradients, and other unknown variables.

(3) Service behavior indicated that slab failure (progressive cracking after development of an initial crack) occurred much more rapidly in slabs on low strength subgrades than in those on high strength subgrades.

(4) Expansion joints were a definite source of weakness unless load transfer devices were employed.

(5) Small slab size (10 by 10 ft) proved less desirable than larger slabs (12.5 by 20 ft) from the standpoint of developing surface roughness under traffic.

(6) Steel reinforcing delayed visible initial cracking and prolonged useful life after cracking.

(7) Crack patterns in the base slab of a rigid overlay pavement are quickly reflected into the overlay slab under conditions of overload.

Revised Criteria

Based upon these findings, revised rigid pavement design criteria were developed using the Westergaard analysis for edge stresses assuming (based on

the Lockbourne tests) that properly designed joints would provide a 25 percent load transfer to the adjacent slab. A design factor of 1.3 was included in the criteria to account for load repetitions of 5,000 coverages.

Lockbourne No. 2 Experimental Mat Tests

In 1944, anticipating the forthcoming B-36 aircraft weighing 300,000 lb and having 150,000 lb single-wheel loads, the Lockbourne No. 2 Experimental Mat test program was undertaken. Some 120 individual pavement sections were included involving 9 to 24 in. slab thicknesses, reinforced and nonreinforced pavements, multiple-layered overlay systems with and without bond-breaking courses, a wide range of subgrade and base course conditions, and a large number of joint design configurations and slab sizes. The tests verified the Corps design curves for this extremely high loading. This also confirmed the ability of the Westergaard equations to predict stresses for such large wheel loads.

While the prototype B-36 was supported on single wheels, production models at the 300,000-lb plus weight, were supported on a dual-tandem (later twin-tandem preferred) wheel landing gear having 150,000-lb on four tires (spaced 31 in. dual by 62 in. tandem). This landing gear change was late in 1945. Plain concrete pavements 12, 15, and 20 in. thick located adjacent to the Lockbourne Experimental Mat Sections were tested under accelerated traffic of the B-36 multiple-wheel loading in 1948-1949. The results again supported the ability of the Westergaard equations to predict stresses in rigid pavements, this time for multiple-wheel loadings.

Results of Lockbourne No. 2 Tests

Some of the more pertinent findings of the Lockbourne tests are listed.

a. Bond-breaking courses between (rigid) pavement layers, even as thin as asphalt prime coats, greatly reduce the useful life of a rigid overlay system.

b. Reinforcing steel in rigid pavements permits a nominal reduction in the required nonreinforced slab thickness. However, the price of steel at the time of the tests made it extremely doubtful that the reinforced design would be economical. As a general rule, this is a valid conclusion today, although special conditions often make the use of reinforcing steel mandatory.

c. Based upon the performance of the test items in Lockbourne traffic tests and upon instrumentation measurements of deflections and strains, the following listing of joint types from strongest to weakest can be made:

(1) Doweled contraction joint.

(2) Doweled construction joint.

(3) Keyed construction joint with tie bars.

(4) Contraction joint.

(5) Keyed construction joint.

(6) Doweled expansion joint.

(7) Free edge expansion joint.

d. There was no apparent advantage to using structural shapes in preference to conventional round dowels for load transfer at joints in thick concrete pavements.

Results of the Lockbourne No. 2 tests, both the experimental mat study and the modification multiple-wheel study, were reported in 1950.

Pickett and Ray Influence Diagrams

In 1950 Pickett and Ray presented influence diagrams for design of rigid pavements in the "American Society for Civil Engineers" Proceedings. They presented influence charts for center and edge loading based on Westergaard's equations and for central loading on an elastic subgrade.

Initial Overlay Criteria

Growth of the B-29 and advent of the B-36 led to a need for strengthening existing pavements. In 1946 a limited investigation (Lockbourne No. 3 Pavement Overlay Investigation) was conducted to study the performance of nonrigid (asphaltic concrete and in some cases, base) overlay on rigid pavement. Based on these limited tests, tentative design criteria for nonrigid overlay on rigid pavement were developed and published in a July 1951 engineer manual.

Model Studies

By 1948 the Lockbourne tests and other early work led to the use of Westergaard edge load criteria, verification of these criteria, provision for multiple-wheel loads and higher tire pressures (200 psi), and provision for empirical supplements to treat repetitions and other factors not treated directly by Westergaard. Small-scale model study techniques had been developed and employed to further verify and extend test section and prototype behavior findings. Model development had begun in the mid 1940's and by the early to mid 1950's substantial dependence was being placed on model studies

for extensions and verifications of design criteria and special elements of pavement behavior. Model studies were used to show that critical stresses in a rigid pavement result from edge loading rather than from corner loading.

Loading Criteria

In 1950 the US Air Force and Corps of Engineers jointly established a loading criteria for the design of military airfield pavements to be used by the Air Force. These were 25,000 lb on a single wheel having 100 sq in.-tire contact area for jet fighter aircraft and designated "Light-Load Pavements" and a "Heavy-Load Pavement" design of 100,000 lb on twin wheels spaced 37-1/2-in. apart and each tire having a 267 sq in.-contact area. These heavy-load criteria were in anticipation of the B-47 aircraft.

There was no anticipation of special or unusual problems from the B-47 aircraft, and criteria based on established knowledge and anticipated loads were published in a 1951 engineer manual.

Effects of High Subgrade Modulus Values

During the period 1951-1954 the condition survey program that began in 1946 was yielding significant findings. The maximum allowable subgrade modulus of 300 pci in design criteria was found to be too low. Following some plate-bearing test studies a correction was made for bending plates, and the allowable maximum k value was increased to 500 pci. The bending correction depended only on foundation strength regardless of the type material involved. The survey program led conclusively to the finding that rigid pavements were performing much better than design criteria would indicate. It was found that while first crack distress occurs about the same time for low strength and high strength support, a rigid pavement on high strength subgrades does not rapidly deteriorate after its first crack, but continues to perform for a greatly extended life. Pavement thickness on k values above 200 pci was reduced to take advantage of this increased life (1954 engineer manual criteria).

Development of Overlay Criteria

The strengthening of rigid pavements by overlay was recognized early and made a part of the test section studies conducted. From tests at MacDill Field, Florida (1944), Maxwell Field, Alabama (1944), Lockbourne No. 1 (1944), Lockbourne No. 2 (both "Experimental Mat" and "Modification" in 1946 and 1949) through the three Sharonville test programs up to 1955, testing was included which permitted the development of overlay thickness design criteria.

This extended to rigid overlay of rigid pavements, including fully bonded, partially bonded, and unbonded overlays. Treatments of acid etching for bond as well as new epoxy bonding were covered for full bonding. Bond breaking treatments were covered for unbonded overlays. The criteria developed also covered reinforced rigid overlays. The Sharonville studies added criteria for flexible overlay of rigid pavements, including bituminous overlay and flexible structure with base and surfacing.

The design criteria developed are well covered in an American Society of Civil Engineers, Aero-Space Transport Division Journal paper entitled "Strengthening Existing Airport Pavements" written by Hutchinson and Wathen.

The "Heavy-Load Test Tracks" at Sharonville constructed in 1957 to study an anticipated 325,000-lb twin-tandem gear loading included an overlay study of pavement already overlaid. This was a rigid overlay of a rigid pavement having a flexible overlay. The results led to the suggestion that the overlay be designed two ways and the more conservative employed. One criterion was to consider the base pavement and overlay as an equivalent rigid pavement and add the further overlay. The second criterion was to consider the flexible overlay layer, even if of substantial thickness, to serve only as a bond breaker.

Channelized Traffic Effects

By early 1954 the B-47 aircraft had been in service about 2 years and pavements (both rigid and flexible) sustaining operations of this aircraft were showing distress in a number of cases. The distress on rigid pavement was in the form of longitudinal cracking near the center of the middle slab of 75-ft wide taxiways. Cracks were continuous for several hundred feet. This condition was soon identified as resulting from channelized traffic and led to a series of field studies which showed that traffic was being applied in critical areas at a rate about six times that anticipated when the pavements were designed.

The development of steerable nose wheels, painting taxistripes for pilots to follow, and a greatly increased ease of preparation for flight (compared to the B-36) had all come together unexpectedly to substantially increase the rate of application of stress repetitions. The result was a shortening of the intended 10- to 20-year life of a pavement to only 2 or 3 years.

Role of Load Repetitions Recognized

Early practice was to design for support of the limiting stress of an established loading. Recognition of fatigue or repetitions as a design parameter in relation to loading had been maturing. The channelized traffic experience, while it tended to confirm the 5,000 coverages previously considered to represent a pavement adequate for unlimited traffic (at the design loading), led to a concensus that load repetitions and load magnitude must be considered in combination for design and evaluation purposes.

Porpoising of Bicycle Gear Aircraft

Concurrent with the channelized traffic problem, it was found that pavement surface irregularities could induce a longitudinal rocking of B-47 (bicycle gear) aircraft. This was called porpoising of the aircraft and led to the introduction of a 15 percent impact factor in loading.

Revised Heavy-Load Criteria

While channelized traffic represented a substantial increase in load repetitions in a critical lane of a pavement, it also represented a narrower lane or band of such critical requirements. In less controlled areas repetitions were more widely distributed and thus less intensive, and it was found that some areas were regularly subject to less aircraft passes or to only aircraft at reduced loadings. This new complexity in areas of different structural design (beyond merely reduced runway center sections) led to the introduction of Type A, Type B, and Type C traffic areas. Type A was to provide for channelized traffic at full intensity. Type B was the prior full design. Type C was for the reduced runway center section, but was extended to areas of only limited repetitions or aircraft loading.

The following heavy-load criteria, based on the various analyses and findings, were placed into effect in May 1955.

a. Design loading. A 100,000-lb load on twin-wheel landing gear having wheels spaced 37-1/2-in. center-to-center and each tire having a 267-sq in. contact area (190 psi inflation).

b. Impact loading. Equal to 15 percent of the gear loading and added to the gear loading for design.

c. Design coverages. Thirty thousand coverages in channelized traffic areas designated as Type A traffic areas. Five thousand coverages in non-channelized traffic areas designated as Type B traffic areas. Thickness of

pavement for 5,000 coverages but reduced by 10 percent in areas of reduced traffic or loading designated as Type C traffic areas.

(1) Type A traffic areas were 50-ft centerstrips of primary taxiways and the first 500 ft at runway ends.

(2) Type B traffic areas were other than centerstrips along Type A areas, secondary taxiways, the second 500 ft at runway ends, warm-up aprons, hangar aprons, and washracks.

(3) Type C traffic areas were runway interiors (other than the 1,000 ft ends), parking aprons, and calibration hardstands.

Channelized Test Tracks

It was still accepted (1955) that the Westergaard analysis supplemented by the small-scale model studies represented a valid basic pattern for behavior, but the fatigue response at the higher (30,000) coverage level required examination. This led to two full-scale test tracks, the Sharonville channelized test tracks, one on low strength subgrade and one on high strength subgrade. Each track included both plain and steel reinforced concrete test items.

Construction was completed early in 1956 and traffic testing was conducted. The results confirmed the extrapolation of criteria for 30,000 coverages (from 5,000 and represented by 12 percent increase in thickness) to represent channelized traffic. These tests included reinforced items recognizing a need for alternatives to the substantial slab thicknesses (up to 22 in.) now indicated for design.

Results of Reinforced Concrete Tests

The following conclusions were stated in relation to steel reinforcement.

a. No reduction in thickness shall be allowed for less than 0.05 percent steel.

b. No reduction in thickness beyond that for 0.5 percent steel shall be allowed regardless of the higher percentage used.

c. All longitudinal construction joints shall be doweled.

d. All transverse contraction or expansion joints shall be doweled.

e. The reinforcement steel shall not extend through any joint of an overlay with the exception of longitudinal dummy joints which may be required to match a joint in the base pavement.

f. The minimum thickness of reinforced rigid overlay shall be 6 in.

g. The maximum distance between transverse jo[illegible] [illegible]orce[illegible] [illegible]e-ment on natural subgrade or base course shall not excee[illegible]

h. The percentage of steel used shall be the same both [illegible]nsversely and longitudinally.

i. The reinforcing steel shall be placed at a depth of 1/4 h + 1 in. from the pavement surface where h is the thickness of the pavement.

j. Reinforcing does not materially affect the number of load repetitions required to produce the first crack in a concrete pavement.

k. The rate of progression of cracking after an initial crack is much slower in reinforced pavement than in nonreinforced pavement.

l. The cracks developed in reinforced pavement are held tightly together.

m. Nominal amounts of reinforcement in concrete pavements increase their useful life and may be used to reduce the thickness of concrete within limits.

n. There is no advantage in placing reinforcement, in the amounts considered, near the bottom of a pavement slab.

o. Subgrade moduli in excess of 200 pci provide benefits by increasing pavement life over and above that indicated by the Westergaard theory.

p. The thickness of concrete for airfield pavements can be reduced in the major areas of runway interiors and aprons by as much as 12 percent over that required for primary taxiways and runway ends.

Results of the Sharonville channelized test track and earlier Lockbourne studies relating to reinforced concrete pavements were the basis for development of empirical design criteria. These were issued in a engineer manual in March 1957.

Prestressed Concrete Pavements

The same concerns that led to interest in reinforced concrete applied even more strongly to prestressed concrete pavements, and by the mid 1950's the technology for prestressed pavements had been slowly accumulating for about 10 years. Theoretical and model studies had been undertaken on a limited basis by the mid 1950's, and in 1956 a prestressed overlay test mat was placed and tested at the Sharonville test site. Later the same year a prestressed slab on grade was constructed at Sharonville. In the meantime, a series of small-scale model studies were carried out to study prestressed concrete pavements. This work resulted, in 1958, in tentative design criteria for prestressed pavements. Only full scale field tests, however, would

provide verification of fatigue or load repetitions behavior, and a test track for prestressed concrete pavements was constructed at Sharonville in 1957.

The Sharonville prestressed test tracks were completed and tested in 1958, and a design procedure for prestressed airfield pavements was formulated. Verification was provided by successful construction of a taxiway section at Biggs AFB, Texas in 1959. A Highway Research Board paper in early 1961 covers development of the prestressed design procedure.

Heavy-Load Test Track

In the late 1950's a further increase in aircraft load was contemplated, and test sections were constructed for a 325,000-lb twin-tandem loading representing a 700,000-lb aircraft. These were the Sharonville heavy-load test tracks. These test tracks were subjected to traffic sufficient to gain verification of criteria extended to cover this magnitude of loading for plain concrete, plain concrete with cement stabilized (then termed lean concrete) bases, reinforced rigid overlays, and prestressed concrete.

Unfortunately, concern for follow-on heavier aircraft was waning in the face of missile developments, and the Sharonville heavy-load test tracks were not exhaustively tested. Findings were subject to review by staff and consultants and were considered in shaping or continuing criteria in engineer manuals, but test results have never been formally reported.

Redirected Studies

Having provided (late in the 1950's) for structural design of rigid pavements and strengthening overlays for up to 700,000-lb aircraft, and with emphasis on alternative strategic defense systems, it was decided to reorient investigational programs toward other aspects of pavements. Thus, through much of the 1960's major emphasis was toward joint sealing materials, improved methods for bonding concrete for repair and strengthening, and general studies of materials and procedures for maintaining existing pavements.

Proof Test Sections

Some mention should be made of the Kelly and Columbus "Proof Test Sections." These were primarily concerned with flexible pavements (see the section on "Flexible Pavement Design Methodology" for more detail) but involved accelerated traffic testing of rigid pavement items. Tests at Kelly AFB in Texas (1956) involved simulated B-47 aircraft traffic at 30,000 coverages. Tests at Columbus AFB in Mississippi (1958) involved simulated B-52 aircraft traffic at the 30,000 coverage level. Both of these tests provided further

confirmation of the rigid pavement design methods as extended to provide for channelized traffic. No revisions were required as a result of the tests.

Rigid to Flexible Juncture

The Kelly and Columbus tests did contribute to improvement of designs for the juncture between rigid and flexible pavements. The Corps had devised a "buried slab" juncture in which a doweled PCC slab extends under a tapering section of flexible pavement. A 1962 ASCE Air Transport Journal paper by Hutchinson and Wathen gives a detailed sketch.

Criteria Adjustments

Some other criteria adjustments, impacting less directly on structure design, were introduced in the late 1950's. The B-52 aircraft had increased in weight to 498,000 lb. The Air Force had introduced an aircraft dispersal concept for B-52 deployment, which reduced the design coverage level. Heavy-load airfield widths had been increased from 200 to 300 ft. Advantage was taken of the central concentration of traffic on runways, which meant few coverages on the edges, and thickness of the outer one-third of runways was reduced and designated as Type D traffic area.

The revised criteria introduced in February 1958 were as follows for heavy-load pavements:

a. Design loading. 265,000 lb on a twin-twin landing gear (37"-62"-37") having 267-sq in. contact area per tire.

b. Impact loading. 15 percent.

c. Design coverage levels.

(1) Type A traffic areas were 10,000 coverages.

(2) Type B traffic areas were 5,000 coverages.

(3) Type C traffic areas were 5,000 coverages at 75 percent of the established design load.

(4) Type D traffic areas were 200 coverages at 75 percent of the established design load.

Documentation of Criteria/Basis of Design Described

A 1966 paper by Hutchinson (ORDL MP 5-7) comprehensively describes the, "Basis of Rigid Pavement Design for Military Airfields". The paper covers traffic loading, ESWL, soil modulus, critical stresses, load transfer, plain concrete, reinforced concrete, overlays, and prestress.

Roads and Streets Design

Also, early in the 1960's there was a requirement for rigid pavement thickness design for roads and streets on military reservations. Methodology was formulated from existing airfield pavement technology and published in July 1961 as Technical Report 4-18.

Multiple-Wheel Heavy-Gear Load Tests

Late in the 1960's wide-body many-wheel aircraft were coming into use and their effect on pavement design was in question. Accordingly, the multiple-wheel heavy gear load (MWHGL) pavement tests (1971) were undertaken. These involved both flexible and rigid, also rigid with flexible overlay, test sections on a carefully prepared low-strength subgrade. In addition to extensive instrumentation studies the test pavements were subjected to traffic of the C-5 aircraft gear and the B-747 aircraft gear.

Findings from the rigid pavement and nonrigid overlay of rigid pavement test items (1970) confirmed that established design criteria could be extended to the new loadings and gear configurations, and used directly without change.

Keyed Longitudinal Joints Found Inadequate

Earlier indications from the Sharonville heavy-load tests that keyed longitudinal joints may not be adequate for the very heavy multiple loads were strongly confirmed by the MWHGL results. Engineer manual criteria were revised to no longer permit keyed longitudinal joints in heavy-duty pavements. A series of tests were conducted to assess various means for strengthening keyed longitudinal joints in existing heavy-load pavements. These are covered and rated in WES Miscellaneous Paper S-72-43 issued in August 1972.

Additional Studies

During the 1960's, several aspects of joint behavior were studied including means for estimating the use-life (or remaining life) of rigid pavement, effects of stabilized foundations beneath rigid pavements, structural aspects of sawkerfs and bored recesses in rigid pavements, cracking in pavements attributable to vibration of the concrete, and deterioration of concrete along joints ("D" cracking).

Transfer of Laboratory Functions

In 1969 the staff and functions of the Rigid Pavement Laboratory at Cincinnati were relocated to the newly established Construction Engineering Research Laboratory near the University of Illinois. Most of the rigid

pavement research functions were further combined with the Corps' flexible pavement research functions in Vicksburg, Mississippi at WES in 1971.

Continuously Reinforced Concrete Pavement

Continuously reinforced concrete pavement was first used on airfields at Chicago's O'Hare field in 1966 and on six other O'Hare and one Midway Chicago pavement by 1971. Continuously reinforced concrete pavement was placed as an overlay on an apron at Patuxent Naval Station in 1968 and on a pavement at Palmdale, California, US Air Force Plant 42. This introduced an interest in and need for guidance for military applications of CRCP. An extensive study was reported in 1974, and design and construction procedures were reported in 1977.

While it was generally anticipated that the steel reinforcement in CRCP would allow a reduction in design thickness, it was ultimately found that the thickness needs to be the same as for plain concrete. The advantage of CRCP is in a freedom from joints and joint problems and in greater smoothness than can be maintained in jointed pavements.

Fiber-Reinforced Concrete

Following promising evaluation of concrete reinforced by fibers placed directly into the mix before placement, field trials were undertaken in 1972 on an overlay placed for the FAA under WES supervision on a taxiway at Tampa International Airport, Florida. Input guidance was provided by the FAA, the Construction Engineering Research Laboratory, and the Greiner Company. Sections of 4- and 6-in. overlay were included. These trials established the feasibility of constructing fiber reinforced concrete pavements for airfields using conventional construction equipment and methods. In 1974 WES reported on steel fiber-reinforced concrete applications to airport pavements.

The design of fibrous concrete pavement is based on limiting the ratio of flexural strength to maximum tensile stress at the joint to a value found to give satisfactory performance. The design is further controlled, however, by a limiting deflection criteria.

Fibers other than steel have been used for reinforcement but experience on which current engineer manual guidance is based is limited to steel fibers. Manual criteria are therefore limited to use of steel fibers, and Headquarters, US Army Corps of Engineers (CEMP-ET) approval is required for applications.

Slipform Paving

During the 1960's the practice of slipform paving was introduced, and by 1970 it was first employed by the Corps on an apron at Luke AFB. The fiber concrete trials at Tampa International used slipform paving. These and other trials and studies led to acceptance of the method for use on Corps projects. A study by F. Parker Jr. of WES reported in 1975 provided the basis for Corps acceptance.

Roller-Compacted Concrete

In 1976 initial trials were seen at WES of what has since become known as roller-compacted concrete (RCC). This first trial was a compaction study of zero-slump concrete. Conclusions from the first study were as follows.

a. Dry-mix concrete can be satisfactorily compacted by heavy vibratory rollers.

b. Satisfactory placement can be accomplished using a conventional base course spreader or asphalt finisher.

c. Strengths comparable to that of conventional wetter (1- to 2-in. slump) mixes can be attained using less cement.

d. Smoothness and surface texture should be sufficient for wearing surfaces of secondary roads and streets and entirely satisfactory as base for other surfacing.

e. This process offers substantial potential for construction cost savings.

f. The cracking from drying, shrinkage, or subsequent expansion contraction is less than for wetter conventional mixes.

More Recent Design Experience Not Yet Reported

Further findings and developments impacting on the design or evaluation of rigid pavements or strengthening overlays are (and will be) covered by currently available references. When sufficient time has passed to provide look-back perspective, this documenting of the background basis of criteria should be updated.

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CHAPTER 6

RIGID PAVEMENT TESTS AND MATERIALS

Modulus of Subgrade Reaction

The early adoption (1942-43) of Westergaard's work as a basis for design of rigid pavements presented the need for a means of establishing the coefficient of subgrade reaction (k) as an input parameter to design computations. The basic Westergaard assumption in his analyses is that the vertical stress providing support beneath a rigid pavement is directly proportional to vertical deflection. In concept this is a surfacing layer having bending strength and supported on a spring-like or dense liquid system representing the subgrade. The coefficient, k , represents the proportion of stress to deflection as a simple ratio. Since stress has the dimension of pounds per square inch, and deflection (or displacement) the dimension of inches, the coefficient, k , has the dimension of pounds per cubic inch.

The coefficient is variously referred to as modulus of soil reaction, coefficient of subgrade reaction, simply k or k-value, and other combinations of these. Westergaard never presented a method (or means) for determining the k-value. He often made reference to k as an empirical makeshift which has been found in the past to give useable results.

It was necessary, from the initial acceptance of Westergaard's work, to establish means for determining the k-value. Early structural tests (1941-42) at Wright Field applied a static (wheel-load-like) load to prepared slabs using a 30-in.-diam rigid plate and measured the deflection beneath the plate and at points away from the center of load. This provided a determination of the entire deflection basin resulting from the applied load. The deflection basin is a volume displacement resulting from the applied load. The ratio of load to this volumetric displacement provides a k-value with its units of pounds per cubic inch. Subsequent analysis of the Wright Field Test results and similar follow-on tests at other locations showed this method to provide k values yielding stresses comparable to those determined from directly measured strains. This was a simple and direct means for determining k in concept, but it was not functionally satisfactory. This means of determination required an existing pavement on a representative subgrade, neither of which could readily be attained.

The volumetric displacement means for determining k did provide a method for evaluating the k value beneath existing pavements. It has been a valuable method for pavement evaluation and continues to find application.

Field-plate loading tests held the potential for providing a more feasible means of evaluating k, and in 1942 tests using a range of plate sizes on natural subgrade and prepared subbase were conducted at Wright Field. These established that determining k value using 30 in. or larger diameter rigid plates would be satisfactory. Since smaller diameter plates did not give good results and larger diameter plates were less practical, the 30-in.-diam plate became standard.

Adjusting for Field Moisture

For design, it is necessary to have the k value represent the field moisture conditions which will ultimately be obtained beneath a pavement. A means for adjusting an as-constructed k value had been devised based on the ratio of deflections in consolidation tests using the as-constructed condition and a saturated specimen condition. The Wright Field plate test program also provided verification of this procedure.

A somewhat parallel program (1942-43) was undertaken by the Corps at WES with a cooperative effort by the Committee on Sampling and Testing, Soil Mechanics, and Foundation Division, American Society of Civil Engineers. This provided further support for the consolidation test method for adjusting to ultimate field moisture. The verification extended to tests on three substantially different soil types. It also attempted to establish useful correlations with more easily conducted laboratory tests. These were unconfined compression tests, triaxial compression tests, consolidation tests on saturated specimens, and CBR tests. No meaningful correlations were established. For the CBR test, it was recognized that data were insufficient and a correlation might be possible with further results.

The basic procedures for establishing a design k value with a 30-in.-diam plate-bearing test were an outgrowth of the Wright Field and following field-bearing test studies. These continue in effect with only minor modification.

Modifications to Plate-Load Testing

A modification, introduced to represent better transient or short term static loadings and repetition effects of traffic than the long term static loadings used earlier, was to revise seating load practice and change from the prior standard of dividing the load at 0.05-in. deflection by 0.05-in. to

obtain k to applying a standard 10-psi loading and dividing by the deflection at that load. These and some of the earlier developments are presented in an ASTM "Special Technical Publication" (1979) by Philippe (June 1947) published April 1948.

A rigid pavement laboratory report of October 1953 explains a number of additional modifications developed as a result of field testing at Sharonville and theoretical analysis.

a. The plate loading on high bearing value subgrades should be increased in increments above the 10 psi (7,070 lb on 30-in. plate) to 30 psi.

b. The load-deformation curve should be corrected for seating or other factors by drawing through the origin parallel to the straight portion of the directly plotted curve. K will be the slope of this adjustment line.

c. Bending of the 30-in. plate, and even of stacked plates, was determined to have significant effect on higher strength determinations. A correlation curve was developed for correcting k-values for plate bending.

d. A minimum thickness of the leveling sand-cushion beneath the plate was determined to be necessary to avoid deviations found for thick sand layers.

e. The 300 maximum value allowed for strong materials by prior engineer doctrine was increased to 500. This had to be attended by increased accuracy of plate displacement (deflection) determinations (1/10,000-in. dials).

CBR Versus K

The field studies leading to plate test modifications also provided a correlation between CBR and k . A number of such correlations, generally all somewhat different from one another, were in existence and were derived by various research agencies.

Standard Method for K Determination

The "Modulus of Soil Reaction," based on the original selection of the 30-in. diam plate, and including the various (minor) modifications indicated, is described by Method 104 in MIL-STD-621A.

Required also for design using the Westergaard analysis were flexural strength, modulus of elasticity, and Poisson's ratio of the concrete. Tests to examine means for determining these parameters and the effect of beam specimen size on the determinations were made in two series of tests. These were carried out in 1948 and in 1950. Details of the second series and comparisons

to the first series are contained in a 1951 report by the Ohio River Division (Rigid Pavement) Laboratories.

Flexural Strength

Both k and flexural strength of the concrete, f_c , are primary material related parameters for design. Prior to the mid 1950's, the widely recognized 28-day strength of concrete was used for design. Since the tensile stresses developed in flexure of the pavement slabs were critical (center slab initially but at slab edge soon after) the flexural stress, determined from one-third point loading tests on 6- by 6-in. specimens long enough for 18-in. span testing, was employed.

By 1953 it was decided that designs should be based on 90-day flexural stress to take advantage of the strength increase known to be involved. In anticipation of this change, the Corps had a number of division laboratories conduct studies to determine the 28- to 90-day strength increase effective for representative concrete mixes involving local materials. The Ohio River Division Laboratories (Rigid Pavement Laboratory) were tasked to consolidate and review these division laboratory studies. This work reported in June 1954 made the following recommendations.

a. Flexural strength tests. Flexural strength tests for mix design and field control concrete should employ standard procedures and beam specimens having a 6-in. square cross-section, length sufficient for 18-in. span, and third-point loading. Concrete containing aggregate larger than 2-in. nominal size should have the oversize particles removed before casting specimens. Specimens are to be cured by submergence in water at the specified temperature for at least 24 hr and maintained in a moist condition until testing.

b. Number of test specimens. For mix design studies a minimum of nine specimens should be tested to determine flexural strength at each test age, preferably by testing three specimens from each of three separate batches of the same concrete mix. For field control tests a minimum of four test specimens should be cast for each test group, two each for tests at ages of 14 and 90 days. All results should be included in determining the average flexural strength for a group of specimens, except that clearly faulty results should be rejected.

c. Mix design studies. When possible, studies should use cement and aggregate proposed for project use to determine the 90-day flexural strength for design. The strength at earlier ages to establish percent strength gain

for use in control tests of specimens was tested earlier than 90 days. When such is not feasible, flexural strength gain may be estimated from the following tabulation.

Age	Percent of 28-day Strength
7 days	82
14 days	91
28 days	100
90 days	110
1 year	119

d. Maximum size of coarse aggregate. Additional data on effect of aggregate size on flexural strength should be evaluated to determine whether guidance should be revised to limit maximum size to nominal 2 in.

Current engineer manual guidance generally follows these determinations, except that large-size aggregate is handled by using larger beams for flexural stress determination. The beams to be used must be square in cross section and have a width at least three times the nominal size of the largest aggregate. Beam length must be adequate to accommodate a span length three times the width.

Design Factor

The design factor relates the design flexural stress from flexural strength tests to the Westergaard maximum free edge stress. It was determined in mid 1940's that the simple loading condition represented by the Westergaard analysis needs to be adjusted by an additional increment to provide for stress repetitions (fatigue), temperature gradients, and other unknown variables. The initial design factor was 1.3, where

$$\text{Design Factor} = \frac{\text{Design flexural strength of concrete}}{\text{Maximum free edge stress}}$$

With the advent of channelized traffic in the mid 1950's it was found that the increase in stress repetitions or fatigue requirements dictated a need for an increase in the design factor. A design factor of 1.54 was used for 25,000 coverages. The 15 percent factor for dynamic effects of bicycle gear aircraft (see the section on rigid pavement design) was provided by increasing the design factor. Design factor values of 1.5 for 5,000 coverages and 1.77

for 25,000 coverages were used for this purpose. The comprehensive American Society of Civil Engineers, Air Transport Division paper by Sale and Hutchinson in July 1959 and the "Basis of Rigid Pavement Design for Military Airfields" paper (ORDL, Miscellaneous Paper 5-7) by Hutchinson in May 1966 provide further explanation of design factor developments and their relation to other design concerns.

Various Developments

A number of technological advancements have impacted on concrete pavement technology through the years, and several such significant advancements are worthy of comment.

Air Entrainment

Air entrainment was a revolutionary advancement in concrete technology. Early developments reach back to the late 1930's, but Corps developments proceeded through the mid 1940's. A 1947 Bulletin (No. 30) by WES, then a part of the Mississippi River Commission, gives some background and state of knowledge at that time. Airfield pavement applications and manual guidance for such applications came in following years.

Curing Compounds

Curing compounds began to be used on concrete pavements to replace wet curing with burlap, paper, or equivalent moisture retaining covering in the mid 1940's. Corps testing began about 1942, produced criteria in 1943, conducted modifications in 1949 and introduced pigmented compounds in 1952. A WES Technical Memorandum No. 6-385 published in June 1954 presents more detail. The curing compound development and acceptance qualification program have continued at WES.

Alkali-Aggregate Problem

The alkali-aggregate reaction problem of deleterious expansion of concrete began to be understood in the early 1940's. Corps examinations and treatments extend through the early to mid 1950's. A May 1956 Miscellaneous Paper No. 6-169 by WES explains the problem at that time. Current standard practice suggests tests to identify potential reaction, to avoid use of problem aggregates where possible, and to use low-alkali cement when poor aggregates must be used.

Pozzolons

Pozzolonic and other materials that can be used to replace cement in mass concrete were the focus of a Corps study undertaken in 1950. Applications

were mostly to dams, and the study was reported in December 1957 as WES Miscellaneous Paper No. 6-247. The economies involved became applicable to airfield pavements when demands of heavy aircraft led to pavement thicknesses exceeding about 20-in. The current "Standard Practice for Concrete Pavements" allows for 25 percent by volume substitutions, greater percentages requiring special testing and approval, but warns that the expected strength at 90-days age may require somewhat longer to mature.

Epoxy Resins

Epoxy resins for bonding to hardened concrete found rather broad trial applications in about 1955 and 1956 and many Corps pavement applications in 1957 to 1959. A 1959 WES Technical Report No. 6-521 concludes that epoxy resins can be successfully used to bond hardened concrete to hardened concrete, bond freshly mixed concrete or mortar to hardened concrete, seal cracks, fill voids, patch popouts and spalled areas, reface worn concrete, protect concrete from chemical attack, and serve as a general adhesive to fasten various items to concrete such as dowels, reflector buttons, and traffic bars.

Pavement Joints

Rigid pavement joint criteria have their origins in the Lockbourne tests, primarily Lockbourne No. 2, conducted in the mid 1940's. These were supplemented and extended to overlay design by the Lockbourne No. 3 tests in the late 1940's and the Sharonville tests in the early to mid 1950's. Greatly supplementing these were the accumulating behavior information from the extensive airfield condition survey and evaluation program.

A 1953-1954 study of sawed contraction joints contributed much of the information on which criteria for this type joint are based. Small scale model studies in 1954 established optimum dimensions for keyed construction joints in rigid pavements. These were dramatically verified by the failure of approximately 50 percent of the male key and 50 percent of the female upper lip of the keyed joint under simulated C-5 and Boeing 747 traffic in the multiple-wheel-heavy gear-load tests in 1971.

A 1956 report "Investigation of Joint Construction in Airfield Pavements" covers results of a survey of joints in airfield pavements for the period 1951 to 1955. Conclusions from this study were as follows:

a. Experience with sawing of contraction joints has been satisfactory.

b. Recommendations from the field for joint requirements generally agree with existing criteria.

c. Sawed contraction joints should be required where practicable. Formed joints should be an option only where conditions indicate that difficulties would be encountered in sawing.

d. A 1/4-in. width of joint opening generally is necessary for effective sealing with jet fuel-resistant sealers. A 3/16-in. width to full depth of the groove may be satisfactory for mild exposure conditions.

e. Widening of sawed joints is not necessary where the minimum required width extends to full depth. A double blade cut to 1-in. depth and beveling of the top of the sawcut are satisfactory methods for widening sawcuts.

f. The depth of groove for dummy joints should be not less than one-sixth of the pavement thickness, but at least equal to the maximum size of aggregate. A deeper groove to one-fourth the thickness may sometimes be required.

g. Spacing of transverse contraction joints should be as follows:

Pavement Thickness inches	Joint Spacing feet
Less than 8	12.5 - 15
8 to 10	15 - 20
More than 10	20 - 25

h. There is little preference between types of sealers, and all types specified can be used satisfactorily.

i. Pressure sealing equipment should be required for contraction joint sealing.

j. Transverse expansion joints should not be required in pavements 10-in. or more in thickness.

Base or Subbase Layers

Through the 1940's slabs were generally placed directly on the subgrade. With the lower repetitions experienced on airfields, as compared to highways, pumping was not a problem, and limited early experience had been interpreted to indicate that subbase or base under rigid pavement had no structural advantage. With the advent of channelized traffic (from B-47 aircraft in early 1950's) pumping was experienced and subbases began to be specified as filter layers.

As the use of base courses continued, manual guidance directed use of the plate-bearing test on top of the emplaced base for determination of the k-value. This practice continued into the 1970's. However, late in the 1950's there was a need for guidance, for evaluation purposes, in assessing the contribution of base course to improve the subgrade k-value without requiring plate tests. Accordingly curves were developed, based on accumulated test section experience supplemented by the substantial condition survey and evaluation program findings, which related K at the surface of the base to base thickness and subgrade k . These have been included in engineer manual doctrine as an alternate to direct plate testing.

For evaluation purposes in the late 1950's the subgrade was commonly evaluated by CBR or equivalent, and the subgrade k was determined using the maturing but not yet uniquely established CBR versus k correlation curve. The combination of subgrade and base was rated for k-value using the curves developed.

Stabilized Layers

Stabilized layers beneath the PCC were treated for use in design in the late 1950's. Manual guidance did not extend to structural improvement, and reference was to the flexible pavement manual for guidance toward type and formulation of the stabilization to be used. In early manuals (1956-60) the structural aspect of a base beneath the concrete layer was not recognized. By 1970 the structural advantage of base was included and stabilized base mentioned as an option, but no guidance for design was included.

The 1988 version of the rigid pavement manual (TM 5-825-3/AFM 88-6, Chap. 3) recognizes both cement modified and cement stabilized materials as underlayers for PCC. Cement modified layers are treated as base layers, cement stabilized layers are treated as an underslab, and the top layer of PCC is designed by overlay criteria.

Summary of Recent Advancements

Several elements relating of rigid pavements for airfields impacted the technology in the 1970's. Fiber reinforced concrete was tried for both overlays and base pavements (see section on rigid pavement structure). A 1974 WES report by Hoff presents information on the materials being used. Slipform paving was employed for airfield pavements in the mid 1970's with attendant problems of edge-slump, key joint forming, placement of tie-bars and dowels, and blockouts for centerline lighting. Attendant problems of jointing, grade

control, and roughness needed to be worked out before the methods could be generally employed on high-quality pavements. In the later 1970's roller compacted concrete (RCC) was introduced involving zero-slump concrete placement using asphalt paving equipment, roller compaction, and vibratory roller compactors.

Rigid Pavement Tests and Materials

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CHAPTER 7

TRAFFIC AND LOADING

Introduction

The greatly increased need for military airfields came with the growing national defense concern just prior to World War II. The Corps responsibility was assigned late in 1940. The B-17 and B-24 bombers were in production and each type weighed nearly 80,000 lb.

Original Loadings

At the outset, the greatest loading for which design criteria for pavements had been developed was a 12,000-lb wheel load and was for highways. There was an immediate need to provide for the nearly 40,000-lb wheel load of the B-17 and B-24 aircraft.

Limiting Loading

Virtually all early concepts were that a certain (heaviest) load to be sustained by a pavement would dictate the design capacity of the pavement structure. There was some recognition (early 1940's) that light duty pavements required somewhat less structure than would heavy duty pavements. It was not, however, clear whether the light duty pavements were subject to smaller loadings, less use, or a combination.

Acceptable Repetitions

Experience with in-service pavements, and more specifically with accelerated traffic test pavements, showed that pavements which did not fail on first loading would be subject to failure under repeated applications of the same load. Test items of various (stair-stepped) thicknesses failed under accelerated traffic at successively larger numbers of repetitions.

Impact Loadings

In the early 1940's, there was concern for the dynamic aspects of aircraft touchdown impact. Common practice was to introduce an impact factor in bridge design where spring action of heavy vehicles is considered to contribute inertial effects adding to static weight. It appeared logical to expect heavy aircraft impacting from above to contribute even greater inertial effects.

Specific tests to examine the impact problem were conducted at Dayton Municipal Airport in 1941 using a Martin B-26 Medium Bomber. These showed

that on normal landings the load at touchdown was only 40 to 60 percent of the static load. Intentionally hard landings could produce loads in excess of the static load, but continuous operations were likely to produce such loadings in only a small percent of landings. Even when they occurred, no two were likely to be in the same location; so cumulative effects were not a concern.

Traffic Effects

It was soon found that early signs of pavement distress invariably were in areas of high concentration of taxiing traffic. This led to the conclusion that repetitive and slowly moving traffic applied the most severe loading. Subsequently, the recognition of more rapid movement with attendant lift and less channelized tracking led to the practice of reducing the thickness of runway interiors.

In applying traffic to test pavements it was recognized that confining all load passes to a single tire-width lane would be unrealistic and unrepresentative. The applied traffic was distributed over a lane chosen to better represent the distributed traffic of actual operations. Since random distribution would not yield known passes over a particular point, the test traffic was carefully applied in adjacent wheel paths to cover the designated traffic lane. This led to the term coverages to designate the number of passes over a particular point on the pavement resulting from test traffic. In 1943 the number of coverages established as being representative of a 10-year or greater pavement life was 5,000.

At that time it was generally accepted that a pavement which could sustain load repetitions (coverages) into the thousands by a particular load would support essentially unlimited repetitions of that load. This concept was more applicable to flexible pavements but was tacitly held for rigid also.

Design Factor

The application of Westergaard analysis to rigid pavement design inherently compared induced stress to strength and implied the potential for failure from a single overloading. For rigid pavement, a design factor of 1.3 was introduced. This provided for a pavement strength 30 percent above the induced flexural stress considered in design. This factor was considered to provide for temperature gradients, slab warping, nonuniform support, fatigue, and other unknowns.

Design Wheel Loads

The B-29 bomber was developed in 1943 and weighed about 120,000 lb. Design criteria were extended from the original 12,000 lb to 25,000, 40,000, and 70,000 lb single-wheel loadings to accommodate the 80,000 lb B-17 and B-24 aircraft and the forthcoming 120,000 lb B-29 on flexible pavement. Rigid pavement testings to confirm extrapolations were at 20,000, 37,000, and 60,000 lb single-wheel loads for a similar load range.

The B-36 aircraft was under development (mid 1940's) and would weigh 300,000 lb. Seeking to prepare for the B-36 and further heavier aircraft extensions of criteria, both flexible and rigid were made to support single-wheel loads of 150,000 and 200,000 lb.

Multiple-Wheel Loading

It had become apparent that the trend in single-wheel load magnitude had passed reasonable limits, and the B-29 was finalized with dual wheels on each main strut (spaced 37-1/2-in. c-c.). While a version of the B-36 was produced having single wheels, the B-36 design was quickly modified, and support was by four-tire, multiple-wheel landing gear in a twin-tandem configuration at spacings of 31 in. by 60 in. and having tire contact areas of 267 sq in. Criteria were now needed for multiple-wheel gear loadings.

Pickett's Influence Charts

The influence charts developed by Dr. Pickett were available in the late 1940's, and designs for multiple wheels could be directly derived using limiting flexural stress and other parameters used with single-wheel design of rigid pavements. These required testing and experience verifications which were programmed and accomplished. For flexible pavements an equivalent single-wheel-load method was developed and used to translate established single-wheel load criteria to the needed multiple-wheel criteria. These also required testing and experience verification which was accomplished. This method is covered in both the ASCE, CBR Symposium, and the WES Collection of Letter Reports.

Effective Repetitions

The verification testing of criteria developed for multiple-wheel landing gear configurations led to a diversity in concepts between flexible and rigid pavements in relation to the effective number of load repetitions being applied. Repetitions applied in traffic lanes by test loadings continued to be the wheel paths passing over a particular (most critical) point as these

paths were placed side-by-side to cover (a coverage) the traffic lane being tested. Thus, counting of coverages remained the same for twin wheels as for single wheels concerning either pavement type. However, in regard to the closely following tandem tires of twin-tandem configurations, there was divergence. On flexible pavements the two wheels were considered to induce two load repetitions. Conversely, on rigid pavements, because of the flexural stiffness of the PCC slab, it was considered, despite there being two wheels passing along the identical path, that only a single load (or stress) pulse was involved, and only one load repetition should be counted.

The result of this difference in concept is that the same twin-tandem load traffic will apply twice the coverages to flexible pavement than it does to rigid pavement.

Tire Pressure Loadings

At this point (about 1948) the need to extend pavement design criteria to provide for higher tire pressure became evident. This consideration was more significant for flexible than for rigid pavements, but it was a concern and was treated for rigid pavements as well. The existing criteria were taken to represent 100-psi tire pressure, and criteria were extended to cover 200 psi.

With the B-36 aircraft, the Air Force had adopted the practice of controlling (limiting) tire deflection by establishing a rolling radius for tires used on each aircraft type. A line was placed on the tires and inflation controlled such that this line just touched the pavement. Result of this was a fixed tire contact area, and tire pressure became a function of tire load. The B-36 tires had tire contact areas of 267 sq in. The B-29 retained 100 psi pressure which produced in later versions a 360-sq in. contact area. The B-50 and KB-50, however, at similar gross weight to the B-29, had the same 37-1/2-in. twin spacing but had 267-sq in. contact areas. Tires were the same as used on the B-36.

Less-Than-Capacity Operations

In 1948 there was a requirement for design criteria for military airfields to support active military operations for less than capacity operation. Permanent airfield pavements, considered to be represented by 5,000 coverage use-life, were for capacity operation. Fields in support of military operations overseas, whether troop constructed or existing and adapted to use, might be required for only limited periods so that their use-life would be for lower coverage levels.

Operational categories were established as follows. These can be found in the Collection of Letter Reports.

Operational Category	Expected Use-life	Coverages Use-life	Percent Full Operation Thickness*
Full	2 years	2,000	100
Minimum	6 months	800	80
Emergency	2 weeks	40	50

* Thicknesses for the full operation category were the same as for permanent airfields in the "Zone-of-Interior" (ZI in World War II terminology) or for capacity operation except that bituminous surfacings were of much lower quality.

These less-than-capacity-operation considerations pertained primarily to flexible pavements. Rigid pavements were not excluded, but they were not of such nature as to have application for troop construction and short use-life.

Expedient Airfields

This treatise does not extend to treatment of expedient military airfields, but some of the developments have impacted on permanent airfield pavement criteria and need to be explained.

It should be noted that the 2,000 coverages for full operation and the 5,000 coverages for permanent airfields are essentially equal structurally. This supported the existing concept that a pavement capable of sustaining the 2,000 coverages could be expected to sustain the 5,000 if its surfacing could stand up to the years of weathering.

Impact on Conventional Pavement Concepts

There were two developments made for these less-than-capacity-operational pavement studies which had significant impact on conventional pavement criteria.

a. One of these was the formulation of pass-per-coverage methods of translating actual aircraft operations on in-service pavements to coverages for use in design and evaluation of pavements. These were taken from observations that 75 percent of traffic is confined to the central one-third of 75-ft-wide taxiways and 150-ft runways. The traffic in this central third is uniformly distributed, and two taxiways serve one runway. The developments conclude that:

$$\frac{\text{cycles}}{\text{coverage}} = \frac{200}{N(W)} \qquad \text{(eq 7-1)}$$

where

N = number of wheels on each main landing gear

W = width of tire print in inches

These developments are covered in the Collection of Letter Reports (MP 4-61).

b. The other development was a plotting of thickness, as a percent of design thickness versus logarithm of coverages (or coverages on a log scale). This plot for flexible pavements can be found in the Collection of Letter Reports (MP 4-61). A 1955 example for rigid pavements is included in a Highway Research Board (Vol. 36) paper.

Standardized Design Loadings

In 1950 the US Air Force and the Corps jointly established a unified loading criteria for the design of future military airfield pavements to be used by the Air Force. These criteria were for light-load pavements and for heavy-load pavements. Light load was 25,000 lb on a single wheel having a tire contact area of 100 sq in., and was to represent jet fighter aircraft loading. Heavy load was 100,000 lb on twin wheels spaced 37-1/2-in. center-to-center and having tire contact areas of 267 sq in. for each wheel. This loading was to accommodate one main gear of the forthcoming B-47 jet bomber.

Channelized Traffic

When the B-47 aircraft had been operating only a few years (1954), supporting pavements were experiencing unforeseen distress. Follow-on studies showed that a combination of the two loadings from bicycle gear, a significantly reduced aircraft wander (width of tracking lane), and increased ease of preparation for operation (leading to increasing numbers of flights), were applying load repetitions to the pavements at about six times the rate anticipated for design of the pavements which led to the term "Channelized Traffic".

Criteria for Coverages

By this time the "OCE Curve" from Miscellaneous Paper 4-61 had come into more general use for research analyses on flexible pavements. It was presented more widely in the ASCE "CBR Symposium." The following mathematical expression for this curve had been formulated:

$$t = 0.23 \text{ Log } C + 0.15 \quad \text{(eq 7-2)}$$

where

t = thickness as a decimal fraction (%t/100)

C = coverages of traffic

Recognition of Repetitions Effect

This acceptance of increasing pavement thickness with load repetitions, to and beyond 5,000 coverages, significantly revised earlier concepts, that a few thousand repetitions represented capacity for unlimited repetitions. Thus, while the need to accommodate channelized traffic had not been anticipated, its accommodation for design (6 × 5,000 or 30,000 coverages) appeared a mere matter of increased pavement thickness indicated by the percent design thickness versus coverages curve. Equivalent extensions were applicable to rigid pavements, and subsequent field tests were accomplished which confirmed the extensions to flexible and to rigid pavement criteria.

Impact Loading Criteria

The B-47 aircraft also brought the experience with aircraft (forward to rear) rocking which came to be called porpoising. The unknown dynamic impact of porpoising and its apparent contribution to distress in the field led to the introduction of a 15 percent impact loading introduced for bicycle gear aircraft. True need for this factor was never satisfactorily confirmed and it was eliminated from design when B-47 bombers were no longer in use. The later B-52 bombers were also bicycle gear aircraft and considered to require the impact loading for design, but the B-52 did not show the same tendency to porpoising as the B-47.

Heavy-Load Criteria

To provide for the B-47 aircraft and based on a two wing (90 aircraft) operation per airfield, the following heavy load criteria were put into effect in May 1955.

a. Design loading. A 100,000 lb on a twin-wheel landing gear, twin spacing 37-1/2-in. center-to-center, and 267-sq in. contact area for each tire (190 psi inflation).

b. Impact loading. Equal to 15 percent of the design gear loading.

c. Design coverages. Type A, B, and C traffic areas defined with coverages and loading as follows:

(1) Type A traffic area--30,000 coverages at the design gear load plus the impact loading. Type A traffic areas were primary taxiways and the first 500 ft of runway ends.

(2) Type B traffic area--5,000 coverages at the design gear load plus the impact loading. Type B traffic areas were secondary taxiways, the second 500 ft of runway ends, warm-up aprons, hangar aprons, and washracks.

(3) Type C traffic area--Same design as a Type B traffic area, except that the pavement thickness was reduced by 10 percent allowing for reduced aircraft loading for operations on these pavements. Type C traffic areas were runway interiors (between 1,000-ft ends), parking aprons, and calibration hardstands.

Light-Load Criteria

Equivalent designs for "light load" pavements for fighter type and other aircraft were developed.

a. Design loading. 25,000 lb on a single-wheel landing gear having 100-sq in. tire contact area.

b. Impact loading. None.

c. Design coverages. 5,000 coverages in all cases, but traffic areas as follows:

(1) Type A traffic areas: None.

(2) Type B traffic areas--1,000 ft runway ends, aprons not having well defined taxilanes, taxiways, and all other pavements generally.

(3) Type C traffic areas--Runway interiors and apron areas between well defined taxilanes.

Revised Operational Categories

In the mid 1950's criteria for airfield pavements to support military operations in the field again had an impact on permanent airfield (Zone-of-Interior) traffic and loading criteria. The less-than-capacity operations considered for pavement design definition in 1948 had encountered some revisions. Newer categories had become:

Operational Categories	Expected Life	Coverages Use-life	Channelized Coverages
Capacity	10-20 years	5,000	30,000
Full	2 years	2,000	
Minimum	6 months	700	
Emergency	2 weeks	40	

Pavements to be subjected to channelized traffic could be expected to be subject to coverage levels six times higher than nonchannelized traffic. In recognition that particular values of coverage level were not truly significant and to reduce the complexity of design criteria in troop-use manuals, the following adjustments were made to the operational category criteria (1956).

Operational Categories	Expected Life	Coverages (Use-life) Nonchannelized	Channelized
Capacity	Over 5 years	5,000	25,000
Full	1-2 years	1,000	5,000
Minimum	4-6 months	200	1,000
Emergency	2-3 weeks	40	200

The purpose was to reduce the number of design plots needed from eight to five for the four operational categories and both nonchannelized and channelized for each.

While this military operational category pattern had several minor effects on future criteria (40 coverage over-run design, 200 coverage edge design), the main effect was to establish heavy-load, channelized traffic for Type A traffic areas at 25,000 coverages. The 30,000 coverage level continues to be cited in some instances.

B-52 Criteria

In 1956 heavy load design criteria were updated for the B-52 bomber. While otherwise similar to the 1955 B-47 criteria, the load was now defined as 240,000 lb on twin-twin landing gear spaced 37-, 62-, and 37-in. center-to-center between adjacent wheels. The B-52 tire contact areas remained at 267 sq in. which is the same as the B-47.

By 1957 the Air Force had indicated an increase in B-52 gross weight to 498,000 lb presenting a gear loading of 265,000 lb. They also established a dispersal concept which would limit the number of B-52 aircraft at a base, and this led to adjustment of the heavy-load design repetitions to 10,000. Heavy-load runway widths were also increased to 300 ft, which made reduction of thickness of the outer one-third (100 ft) a more viable option. The concentration of traffic within a confined central lane with consequent low use of edges had been earlier established and recognized. All of this led in early 1958 to the establishment of the following.

a. Design loading. 265,000 lb on a twin-twin wheel gear, having 37-in. spacing of twins and 62-in. spacing between interior wheels, and 267-sq in. contact area per tire (250 psi inflation).

b. Impact loading. 15 percent of the design gear load.

c. Design coverages.

(1) Type A traffic areas. 10,000 coverages at maximum design gear loading plus impact loading.

(2) Type B traffic areas. 5,000 coverages at maximum design gear loading plus impact loading.

(3) Type C traffic areas. 5,000 coverages at 75 percent of the design gear loading plus impact loading.

(4) Type D traffic areas. 200 coverages at 75 percent of the design gear loading plus impact loading.

Type A, B, and C areas were similar to those for the 1955 B-47 (heavy load) criteria. Type D traffic areas were the outer (one-third) edges of runway.

Revised Equivalent Single-Wheel Load Criteria

In the mid 1950's, the design loading applicable for flexible pavements was modified somewhat by revision of the method for deriving equivalent single wheel loads representing multiple-wheel landing gear. Accelerated traffic tests to confirm the initial method had yielded slightly unconservative results. The newer method was based on a reanalysis of available data confirmed by traffic test results.

Lateral Traffic Distribution

In 1957 and 1958 some specific studies of the lateral distribution of aircraft on runways during takeoff and landing were conducted. These were considered necessary to supplement the 1955 channelized traffic studies which defined lateral distribution of traffic on taxiways.

The direct observations of aircraft wander from the 1955 channelized traffic studies, confirmed by the 1957 and 1958 lateral distribution studies, resulted in methods which supplanted the early methods; i.e. passes-per-coverage = 200/NW. N = number of wheels per gear. W = tire print width. These methods are covered in some detail in the Instruction Report S-77-1 by Taboza.

It was found that the lateral distribution of traffic for takeoff runs was about the same as for taxiways. Distribution for landings was wider but

generally well within the central one-third of even the narrower (200 ft) runways.

Army Airfield Criteria

The Army, in about 1957, found the need for Army airfield design criteria. This was, at first, merely the single-wheel criteria extended to the lower wheel load range. First criterion for both flexible and rigid pavement was for single-wheel 100 psi tire pressure and single-wheel 100-sq in. tire contact area. For flexible pavements, single-wheel 50 psi tire pressure curves were also developed, but these were for pavement evaluation and did not appear in design manuals. Curves with 200 psi single wheel were later added to accommodate some helicopters having over 100 psi tire pressures.

By the early 1960's Army aviation had the CV-2 Caribou aircraft, and design curves were issued for twin-wheel aircraft having 20-in. twin spacing and 100-sq in. tire contact areas. Technical manual guidance in 1965 established four airfield classes:

Class	Aircraft	Loading
SM	Skid-mounted helicopters	Class F road design for intermediate traffic.
AA	Army aircraft wheel-mounted	15,000 lb load on twin wheels spaced 20 in. apart and having 100 sq in. contact areas.
LC	Light cargo in addition to Army aircraft	25,000 lb load on single-wheel gear, 100-psi pressure.
M-HC	Medium-heavy cargo (C-124, C-130, etc.)	Gear load of critical aircraft expected.

In about 1965 a conflict of Army and Air Force missions was settled by limiting Army fixed-wing aircraft to 15,000 lb and permitting the Army expanded use of helicopters.

By 1968 criteria for Army aviation had been adjusted to the following:

Class	Aircraft
I	Rotary- and fixed-wing aircraft having gross weight of 30,000 lb or less.
II	Rotary-wing aircraft having gross weight over 30,000 lb.

Class	Aircraft
III	Fixed-wing aircraft with maximum gross weights over 30,000 lb. (These are Air Force aircraft such as the C-130 using Army fields).
IV	Fixed-wing aircraft larger than allowed in Class III. (These are Air Force aircraft such as C-141, KC-10, etc. using Army airfields).

Present criteria contained in TM 5-803-4 are as follows:

Class	Aircraft
I	Rotary- and fixed-wing aircraft with maximum gross weights equal to or less than 20,000 lb.
II	Rotary-wing aircraft with maximum gross weights between 20,001 and 50,000 lb.
III	Fixed-wing aircraft with maximum gross weights between 20,001 and 175,000 lb and having either single-wheel, twin-wheel, or single-tandem gear configurations.
IV	Multiple wheel fixed- and rotary-wing aircraft other than those considered for Class III.

Roads and Streets Criteria

In 1961 revised thickness design methods (WES TR 3-582) for flexible highway pavements were presented. The procedure set a pattern for combining effects of mixed traffic loads to a single equivalent for one selected load. For highways this was an 18,000 lb axle load. Similar combining methods were later applied to aircraft loadings by the Corps for both Air Force pavements and FAA pavements. While these methods were not employed for airfields until somewhat later, and more for airfield evaluation than design when employed, the method is notable here as a basic beginning.

Equivalency Factors

Also notable at about the same time is a research effort attempting to establish some actual equivalency effects of mixed loads on airfield pavements. It is notable because it is the only such effort that, so far as is known, has been attempted. The tests involved basic 10,000 lb load repetitions compared to the basic 10,000 lb load with 10 percent of 25,000 lb overloads. Also attempted was the basic load with 10 percent of the 250 percent overload and 1 percent of 50,000 lb or 500 percent overload. Results, while extremely interesting, were not sufficient to result in specific definable patterns.

The treatment of mixed loadings and load equivalencies remains an undesirably crude procedure, which is sorely in need of improvement. Conceivable

research to improve methods, however, would be impracticably and demanding of available research support and improvements are not likely.

Medium-Load Design

By 1960 a medium-load design for Air Force airfields had been added to the earlier heavy-load and light-load designs. The heavy and light designs remained as earlier. The medium-load design was 100,000 lb on twin-wheel gear having 267-sq in. tire contact areas. This loading originated with the B-47 but accommodated the B-50 and KB-50, the KC-97 and C-97, and was found to satisfactorily provide for the KC-135 and C-135 and later for the C-141. The medium-load design later changed to C-141 at 320 kips and then to 345 kips.

Aircraft Ground Flotation

There is need to mention a 1960 study and earlier lead-in work on what, for want of a better designation, is called "Aircraft Ground Flotation." While this study did not directly determine a loading or traffic definition used by the Army or Air Force, it has had a significant impact on aircraft loadings on pavements.

Ground flotation refers to the strength a particular aircraft type or design requires of a supporting medium on the ground. This becomes a combination of wheel load, number and spacing of tires, and tire pressure (actually ground contact pressure). While the methods of pavement design have formulated means for designing pavement structure to deal with the requirements of individual type aircraft (and loading), there was a need to provide aircraft designers with an indication of the effect on "ground flotation" of variations in wheel load, in tire spacing, and in tire pressure. This requirement needed to be relatively simple to comprehend and be employed by aircraft designers.

Air Force had guidance documents for aircraft designers and needed a means for ground flotation guidance to include in them. In 1952 the Air Force devised (there was some Corps input to the work) the "Unit Construction Index Chart (UCI). The UCI was an index number which attempted to reflect the construction effort required for a flexible pavement to support a defined loading. It was actually based on the quantities of materials to provide a reasonable design for flexible pavement considering a moderately low subgrade strength and a specific length and width of landing strip with no other pavement or attendant concerns. The C-130 and C-124 transport aircraft had limiting UCI values as an element of their design requirements.

Inadequacies of the UCI and requirements for development of the heavy-load-system that led to the C-5 aircraft permitted development of improved ground flotation criteria. The new heavy cargo aircraft was to have capabilities for operation in forward military areas as well as a heavy-lift capability when operating from permanent airfields. A study was conducted of design requirements for permanent type pavements and for expedient type facilities in the theater-of-operations. These extended to single, an array of twin, and an array of twin-tandem spacings for a variety of tire contact areas in each case. A report of "Ground Flotation Requirements for Aircraft Landing Gear" published in December 1961 (revised July 1965) was an amalgamation of the various pavement type and subgrade strength variations. Thus, the criteria presented would permit selection of gear type, contact area, and wheel spacings which could be expected to find satisfactory support by airfield facilities to be used in a large majority of cases.

Perhaps the greatest contribution of this and related work was the acquaintance of aircraft landing gear design engineers with ground flotation parameters and concerns. This had great impact on design of the C-5 aircraft landing gear and appears to have influenced all wide-body aircraft in commercial use. Thus, the loading of pavements by these very heavy (to 800,000 lb and above) aircraft has been greatly influenced by the aircraft ground flotation work.

Estimating Pavement Life

In 1962 the Rigid Pavement Laboratory prepared a methodology for estimating the life of rigid pavement for airfields. This was a formalization of a process which had become recognized as effective for both rigid and flexible pavement. The capability existed for projecting the total use-life in terms of load repetitions (coverages) for an airfield pavement subject to defined using traffic. The capability did not exist to measure any parameter or combination of parameters which would directly indicate how much of the life of a pavement had been used or remained. It was, however, possible to determine the traffic which had used a pavement since it became operational. This involved reducing the mixture of traffic and loadings found for a pavement to an equivalent number of coverages of a selected critical load. By subtracting the traffic already sustained by the pavement from the total it could be expected to support during its effective use, the remaining life, in terms of coverages remaining, could be estimated.

Alpha Factor

Late in the 1960's (1968-1970) the Air Force (AFWL-Kirkland AFB) asked WES to restudy the relation of coverages (load repetitions) to required flexible pavement design thickness in the light of accumulated test experience since development of the relation then in use (%t = 100 (0.23 log C+0.15)). It was this analysis which developed the family of curves of percent design thickness versus coverages. It was found that for more supporting wheels less thickness was needed at the larger coverage level. These curves are called the α curves, and the CBR design equation has been restated as:

$$t = \alpha\sqrt{\frac{P}{8.1\ \overline{CBR}} - \frac{A}{\pi}} \qquad \text{(eq 7-3)}$$

where α is the percent design thickness taken as a decimal fraction for the coverage level of concern with regard to the number of supporting wheels contributing to the design load. A full working set of α-curves will be found in the WES Instruction Report IR S-77-1 dated June 1977 entitled "Procedures for Development of CBR Design Curves".

Mixed Traffic Design

By late 1970's Type A and D traffic areas had been added to the medium load category of airfields, and a shortfield category had been added with the C-130 as controlling aircraft. Recognizing the need to treat a mix of traffic rather than a single maximum load, the following loadings were established by the Air Force in December 1982.

A and B traffic areas

Light Load
1,000,000 passes of the F-4 (60,000 lb)
50,000 passes of the C-141 (345,000 lb)

Medium Load
100,000 passes of the C-141 (345,000 lb)
100 passes of the B-52 (400,000 lb
25,000 passes of the F-4 (60,000 lb)

Heavy Load
30,000 passes of the B-52 (480,000 lb)
50,000 passes of the C-141 (345,000 lb)
25,000 passes of the F-4 (60,000 lb)

Weight-Bearing Capacity

Some comment is pertinent on the publication of aircraft weight-bearing limitations for pavements. It has long been the practice to publish

information on airfields, both US and world-wide, in military Flight Information Publications (FLIP) and in civil Aeronautical Information Publications (AIP). These publications include information on the heaviest loads allowed on the airfield.

Since the early to mid 1960's there had been dissatisfaction with the diversity of means for conveying the limitation imposed. These included gross weight for single-, dual-, and dual-tandem aircraft; gear load for single-, dual-, and dual-tandem, British load classification number, both older and revised. For non-US fields, it was even poorer. These included, designation of a particular aircraft type, single-wheel load, ESWL, LCN, and the S, D, and DT.

Efforts toward a single reporting means were pursued by the International Civil Aviation Organization (ICAO). Beginning in the 1960's and extending through a number of unsatisfactory trials, a study group sponsored by ICAO finally produced an acceptable method which was adopted by ICAO effective November 1981. This adoption commits member states, with the strength of treaty agreement, to adopt and practice the reporting method unless the member state specifically declares a nonagreement (with all or parts of the method). The US has accepted the method and is committed to its use.

This method has become known as the aircraft classification number/ pavement classification number (ACN/PCN) system or method. Both ACN and PCN are represented by a defined single wheel which can vary only in load magnitude. PCN is determined for an airfield in terms of the maximum load to be allowed on the airfield when loaded by the established standard single wheel.

Each type aircraft must have relations (curves or equivalent) showing the aircraft load which has the same load support requirement as the standard single-wheel load. The PCN establishes the largest ACN allowable, and the ACN to aircraft load relations provides the limiting load for that type aircraft.

Because the single-load to multiple-load equivalents are not the same for weak as for strong subgrades, the method establishes four levels of subgrade strength. The strength (high, medium, low, and ultra-low) range applicable to a particular pavement must be used. Within the established strength ranges the equivalence of single-wheel to multiple-wheel loadings is close enough for the ACN/PCN method to function satisfactorily.

Traffic and Loading

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CHAPTER 8

COMPACTION

Compaction Test

When the Corps tentatively adopted the California pavement design method (the CBR method), it was decided that their compaction test (used for test specimen preparation for CBR determination) would be impractical for Corps use. The California method used a 2,000 psi static load to produce standard density in the 6-in. diameter mold. The loading equipment required was not consistent with either field control of compaction for pavements on permanent United States bases or for troop use applications.

Modified Compaction Test

It was decided to use a dynamic or impact compaction test, such as the Standard AASHO or Proctor compaction test. It was also felt that for the California methods to satisfactorily apply for Corps use, it was necessary for a suitable compaction test to produce densities equivalent to those being gained on California pavement constructions by their field control methods. The Standard AASHO Test did not produce high enough density.

This led to the origin of the modified test. It represented a modified compacting effort, and has been commonly termed Modified AASHO Compaction or Modified Proctor Compaction, and subsequently CE-55 Compaction under Military Standard 621. The 6-in. mold and (nominal 5 in.) 4-1/2-in. high specimen were already the standard for CBR specimens. The Standard 5-1/2-lb hammer was changed to the Modified 10-lb hammer. Standard 12-in. drop went to Modified 18-in. drop, and compaction in three layers changed to compaction in five layers.

The extensive laboratory testing carried out in 1942-1944 to examine, adjust, and standardize aspects of testing in relation to determinations of CBR for pavement design purposes included compaction tests. These compared standard effort, intermediate effort, modified effort (now commonly 12, 26, and 55 blows per layer), and the resulting full set of compaction curves. Compaction to 12,000, 26,000 and 55,000 ft-lb/cu ft was exercised and reported. The general difference in compactibility of cohesive and noncohesive soils was recognized, and interestingly the separation between these general soil types was found to have a plasticity index (PI) of 2.

The Modified AASHO or Proctor Compaction test has been used for Corps pavement work since its beginning (1942-1944). While this test method is not directly an element of compaction criteria for pavements, the criteria imposed makes use of this test and of the separation between cohesive and noncohesive materials.

Compaction Requirements for Flexible Pavements

Specific compaction requirements for flexible airfield pavements were introduced through the 1943 Engineer Manual criteria. It was required that the top 6 in. of the subgrade be compacted to 95 percent of modified AASHO maximum unit weight. All fill material below this 6 in. was to be compacted to 90 percent of this unit weight. In cut sections there was no requirement for compaction below the top 6 in.

A 1945 study, although never formally reported, was made of the densities existing in pavements subject to aircraft loadings. This included several accelerated traffic test sections and in-service pavements at the following airfields:

Barksdale	Eglin
Corpus Christi	Grenier
Lewistown	Langley
Natchitoches	Santa Maria

As a result, the following revised compaction criteria were used in a 1946 engineer manual:

	Depth in inches below pavement surface Modified AASHO			
	All but Cohesionless Sands		Cohesionless Sands	
Wheel Load lb	100%	95%	100%	95%
5,000	--	--	--	12
15,000	--	12	12	24
40,000	12	18	24	36
60,000	18	30	30	48
150,000	30	54	48	78

In 1950 it became necessary to extend the compaction requirements to provide for higher tire pressures and multiple wheels. A study showed that the established single-wheel compaction requirements could be consistently related to computed maximum shear stress. These calculations were for a homogeneous half-space (single layer) model using the theory of elasticity. Calculations could be made for the higher tire pressure loadings, and multiple-wheel calculations could be made by superposition of single loads.

The maximum shear stresses thus determined could be used with the shear stress versus percent density correlations to develop compaction requirements for the higher pressures and multiple-wheel loads. These criteria were issued as curves in the 1951 Engineer Manual for flexible pavements.

Compaction Requirements for Rigid Pavements

While not nearly so critical as for flexible pavements, compaction criteria were also of concern for rigid pavements. Early requirements in engineer manuals were for 90 percent modified AASHO maximum density for fill sections other than those composed of cohesionless sand. Cohesionless sands and sandy gravel were to be compacted to 100 percent modified AASHO density in the top 6 in. and 95 percent for the remaining depth of fill. In cut sections the top 6 in., excluding cohesionless and sandy gravel subgrades, was to be compacted to 90 percent modified AASHO density. Cohesionless sands and sandy gravels required 100 percent density in the top 6 in. and 95 percent density in the next 18 in. These requirements have remained in effect in much this form to recent times.

Criteria Modifications

The 1951 compaction criteria for flexible pavements discussed earlier treated 100 and 200 psi tire pressure single-wheel loads and included twin and twin-tandem gear configurations for the B-50, B-36, and the B-47 (then just coming into service). With the different configurations of other twin (C-124, C-118, C-119, and C-121), single-tandem (C-130), other twin-tandem (C-133 and C-135) and the forthcoming B-52 with twin-twin gear, there were additions and modifications to the 1951 criteria.

Compaction Requirements Study

In 1954 to 1958 an in-depth study of compaction requirements for flexible pavements for heavy airfield pavements was programmed and conducted. This study assembled some 150 data points from accelerated traffic tests and over 900 from in-service pavements which represented densities found beneath pavements subject to using traffic. While this was a substantial amount of data, it would not permit separate analysis for the effective parameters of load, tire pressure, number and spacing of wheels, and depth within the pavement, and these separately for noncohesive and cohesive soils. This study introduced the compaction index indicated as either CI or C_i for ready reference which combined all the effective parameters into a single index quantity. The CI could be plotted against percent modified AASHO density. While the

resulting pattern reflects substantial scatter, it also shows a quite strong trend. Some verification of the validity of the CI was gained by applying it to the earlier (1951) compaction requirements with a resulting consistent pattern.

A single criterion for each general soil type, cohesive and noncohesive, was developed from which specific compaction criteria could be readily derived for any specific aircraft loading. the analysis established a separation between cohesive and noncohesive soils at PI = 0. Some Corps criteria continued to be issued using the commonly recognized PI = 5 (generally PI = 6 for others concerned with engineering soil behavior), separation between plastic and nonplastic soils. This analysis provided input to some tentative criteria issued in 1955 in response to the channelized traffic problem encountered (see page 8-4) and has provided the basis for design manual compaction criteria (light, medium, and heavy load) until a further restudy was made in 1986.

Impact of Channelized Traffic

In 1952 as the B-47 aircraft came into service, the channelized traffic problem was experienced. Pavements supporting the B-47 were experiencing distress after about 2 years of use. Examinations of the problem and distressed pavements indicated that airfields were experiencing in about 2 years the traffic which had been expected to be applied in over 10 years. Much of the flexible pavement distress (but not all) appeared to reflect inadequate densities within the structure. This raised a question as to the capability of existing equipment and methods to produce high enough densities during construction to prevent undue additional densification under channelized traffic.

In June 1955 the Corps issued interim criteria for pavements to support channelized traffic of heavy aircraft. These increased base and total thickness also increased compaction requirements. This criterion also introduced proof rolling with a heavy rubber-tired roller on top of the subbase and on each layer of base. Proof rolling involved 30 coverages of a four tire roller having 30,000 lb per tire and 150 psi inflation pressure. Proof rolling is required in Type A traffic areas.

To verify or adjust the interim criteria for channelized traffic, an accelerated traffic test was conducted and reported in 1962. The test section included a deep (120 in) sand item for density study. The test results confirmed the revised compaction requirements, and these with the proof rolling

have been continued. The proof rolling requirements have remained much the same to the present.

Proof Rolling

In concept, proof rolling has two valuable attributes. The 120,000 lb on four tires applied at top-of-subgrade level has a compacting effect equivalent to the effect of loads on the surface which are much larger or applied in many more repetitions. This is the attribute which contributes to providing sufficient density during construction. The second attribute is in relation to the structural design. Proof rolling can bring to light deficiencies in the lower structure which would otherwise lead to early distress under traffic on the finished pavement. Unfound soft layers within the subgrade or construction deficiencies which leave zones of insufficient shear strength will be overloaded by the proof rolling and will be found. Correction during construction is far cheaper than (deep) repairs to the finished pavement. Even where no deficiencies are present, it is reassuring to have the challenging loading adequately supported and be assured that no weak areas will later affect the pavement.

Compaction Requirements Restudy

The compaction criteria reported in 1962 have continued to serve as the basis of criteria for specific design loadings. The means of application of these criteria, however, led to a question which resulted in a restudy of compaction criteria for Corps use in 1986. This document is being prepared to provide background information on Corps pavement design requirements through the decade of the 1970's, and leaving later aspects to currently available references. In this case, however, it is considered pertinent to extend coverage to the 1986 study.

The 1962 criteria represented compaction requirements as a continuous relation (a curve) of needed density to depth. In practice, however, tabulations were included in design manuals which set required depths for selected values (100, 95, 90, 85 percent) of density. The depths listed were selected from the continuous curve criteria. It was not recognized that requiring density in increments to the successive depths (in concept) left the top few inches of a next lower layer at a density lower than the continuous relation of density to depth requires.

This apparent unconservative application of compaction criteria became a question, and problem when computerization of the design manual requirements

were underway. What made the problem was that despite the unconservative treatment there appeared to be no problems with inadequate compaction of pavements in-service for many years. There was, thus, an apparently valid and accepted analysis of compaction needs unconservatively applied but with no resulting insufficiencies. Adjustment to the more conservative pattern would mean increasing construction costs with no apparent need. The alternatives were to ignore the problem or reexamine compaction requirements.

The reexamination was undertaken. Pavement representatives for each military service were canvassed to assure that no low-density distress was being experienced. Informal inquiries to other sources strengthened this finding. Some 400 airfield evaluation reports were searched for density information from in-service US Air Force airfield pavements. These also showed no pattern of distress from low construction density. The additional data and the earlier (1954-1958) study data were combined and reanalyzed. The analysis confirmed that a strong, still with undesirable data spread, pattern of density was required as opposed to depth. The spread or dispersion of the data about the central average curve was consistent along the curve for both cohesive and cohesionless soils. Thus, various (nominally parallel) curves can be drawn which enclose various percentages of the densities found to exist under traffic within in-service pavements. The 80 percent (enclosing) curve has been recommended as a basis for specific compaction criteria generation, as needed, since it is consistent with earlier criteria. At this time (1990) no specific decision on this recommendation has been formalized as a basis for manual criteria revision.

Compaction

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CHAPTER 9

PAVEMENT FAILURE AND TERMINAL CONDITION

Structural Failure

At the outset of concern for heavy airfield pavements in the early 1940's, concepts of pavement distress or, as then commonly termed, pavement failure were widely and commonly accepted. Badly cracked rigid pavements or cracked and distorted flexible pavements were failing or had already failed. There was not then and there is still not a uniquely held consensus of some degree of distress recognizable as failure. Therefore, it was necessary to identify some degree of distress as an end-of-life or terminal condition which is referred to as failure.

Rigid Pavement Concepts

Rigid pavement concepts initially considered that loads less than critical could be sustained with no effect, and loads at or above the critical level would cause cracking of the concrete slab. It was thus accepted that failure of rigid pavement was represented by first apparent cracking.

Flexible Pavement Concepts

Flexible pavement concepts were that pavement distress resulted from loads which caused internal shear movements. So it was accepted that failure was represented by surface indications of internal shear movements rapidly progressing. It was recognized that surface depressions would result from densification under traffic, but that these were essentially independent of internal shear movements from overloading. The processes of providing sufficient structural strength to resist shearing and of providing sufficient density in the various pavement layers to avoid undesirable further compaction under traffic had to be separately assured.

Early Limiting Load

It was accepted virtually without question that a particular pavement would fail if it sustained too large a load, but that smaller loads could be applied freely without limit. Contrarily, it was recognized that in general a pavement would not fracture or rut under a single or some small number of passes of a wheel load large enough to fail the pavement. It was considered that a pavement which could withstand a particular loading for perhaps 1,000 to 2,000 load repetitions would not fail under that load despite many more applied repetitions. It follows that a larger load would produce failure

under less than the 1,000 to 2,000 repetitions and was too large a load for the structure.

First Crack Failure in Rigid Pavement

As accelerated traffic tests were conducted to verify extensions of design criteria, the foregoing concepts were the basis for deciding failure of test items. Ideally, all slabs in a rigid pavement test item should behave the same, but nonuniformity would obviously work to prevent such behavior. It would be unrealistic to decide failure on the basis of the first crack in the first slab to show distress. Failure was considered to be the appearance of a first crack in half (50 percent) of the slabs being loaded (trafficked).

Shear Failure of Flexible Pavement

For flexible pavements the determination of failure was much less specific. It required the recognition of effects of internal shearing. This could be pattern cracking tending to alligator cracking associated with support instability. It could be longitudinal cracking with shallow rutting and attendant uplift outside the wheel-tracking lane. On occasion it could be the shallow rutting with parallel uplift outside the tracking lane but without cracking. In test section work the evidence of internal shearing might only be recognized from the patterns of layer thinning and thickening found from the cross sections exposed on the sides of test pits cut across the traffic lane.

Densification Rutting

Wheel-path depressions resulting from increase in density without significant shear displacements were not considered as distresses leading to failure. In the second Stockton tests depressions of nearly 2 in. were recognized as resulting from densification and the test items were not considered failed. Increase in density actually produced an increase in structure strength and resistance to shearing.

Consistency of Recognized Failure

In examining the source or basis of design criteria in use, the consistency or inconsistency of practices leading to data determinations should not be ignored in considering the validity of extant criteria. Some elements of early practice in relation to failure determination should be noted and considered. Since a test load, being repeatedly applied, was considered to be an overload (failure load) if it resulted in severe distress in less than about 1,000 or 2,000 repetitions, it was not of great importance to consistently

recognize the same degrees of distress in test items considered failed. Also, since satisfactory behavior was considered under 1,000 to 2,000 repetitions of a test load, there may be cases in which failure would have occurred between the level of repetitions at which testing was stopped and the 5,000 coverages established as standard. Familiarity with the test data, however, leads one to believe there are few, if any, such instances. A more significant and less definable consideration is the likely variation in the concepts held by responsible test personnel in failure determinations. Not only are concepts held by those responsible in the 1940's likely to be significantly different from those in the 1960's and 1970's, but also the same individual had reasons to change concepts over 10 to 15 years of developing understanding.

Rigid Pavement on Better Supporting System

In the early 1950's the better performance of rigid pavements on high strength (over 200k) subgrades was recognized. It was found that while the first-crack determination of failure was about the same regardless of subgrade strength, for pavements designed by then current criteria, pavements on high strength subgrades continued to perform despite the cracking. On low strength subgrades, slabs would quickly deteriorate to a shattered condition after sustaining a first crack. On high strength subgrades, however, cracked slabs would continue to support traffic and only slowly deteriorate. In the mid 1950's, in recognition of this phenomena, failure criteria for rigid pavements on subgrades of k = 200 pci or higher were revised to provide for the extended use-life. A shattered-slab failure condition was defined which allowed for slabs cracking into six pieces.

Channelized Traffic

In the early to mid 1950's heavy airfields supporting B-47 aircraft traffic experienced earlier than expected distress and failure of pavements. Rigid pavements experienced cracking, and, while 20-year designs were in trouble in 3 to 4 years under using traffic, the distress was consistent with failure concepts being employed. For flexible pavements, the same was true where shear deformation was the distress mechanism, but much of the channelized traffic distress in flexible pavements resulted from densification under the intensified traffic. It has been earlier pointed out that the prevention of shear deformation and the provision of adequate density in construction to limit the compacting effects of traffic are separate concerns. Much of the channelized traffic effects on flexible pavements were a form of distress or

failure relating to densification under traffic, as separate from concerns for shear failure of the pavement structure.

Functional Failure

The bicycle-gear B-47 aircraft also brought the porpoising problem which began concerns for dynamic effects on aircraft of runway roughness. No specific ultimate condition of runway roughness has been determined to represent a pavement failure, but isolated instances of undesirable roughness have required correction of the problem.

Toward the end of the 1950's and into the 1960's, there was developing concern for skid-resistance and hydroplaning. The AASHO road test analysis introduced the concept of present serviceability index of a pavement. These occurrences along with the roughness led to considerations within the community of pavement behavior engineers and scientists of what was termed functional failure. This was functional failure as distinct from structural failure. A pavement was considered failed when it no longer satisfactorily served the function for which it was developed. These considerations were strong for a few years but have since largely waned. Concerns for roughness, skid-resistance, hydroplaning, and excess noise continue as ancillary pavement design considerations, but the provision of adequate pavement structure continues as a prime matter.

Failure in Rigid Pavements

The recognition of failure in rigid pavements continues much the same to the present. There are concerns for spalling, D-cracking, faulting at joints, cosmetic cracks from late joint sawing or from improper curing, popouts, and disbonding of bonded overlays, but none of these represents structural failure. They can become severe enough to represent functional failure and require extensive repair. Prestressed pavements have a different failure mode than conventional rigid pavements, but after first signs of distress, such as cracking above the tendons, the pavements rapidly deteriorate. Fiber concrete cracks as early as plain concrete, but the fibers hold the cracks closed and use-life can continue to a shattered slab condition. Continuously, reinforced concrete pavement is planned to have a close crack interval so that it cannot be judged by a first crack as failure. The Corps has not used continuously reinforced concrete pavement except in a very few instances, and no criterion has been stated for failure. It would appear, from experience with highways,

that the first sign of cracking normal to the expected cracks would quickly deteriorate in a somewhat similar manner to that of prestressed pavement.

Overlays

Flexible overlays of rigid pavement cannot be considered failed based on either first crack or on internal shear movement. They are subject to reflective cracking from the cracked or cracking base slabs and their joints. Failure is judged on the basis of the developing reflective cracking and continued useability of the deteriorating pavement.

Fatigue Life

The AASHO road test and follow-on pavement behavior studies strongly supported by the interstate highway program have impacted the concepts of behavior of flexible pavements. The highway studies and analyses lead to concern for load repetitions in the millions or far beyond the expected use-life repetitions for heavy military airfields. Failure concepts were accepted for highways based on the fatigue life of the asphalt mix surfaces. The many repetitions at lower pavement deflections (smaller loadings) were resulting in surface cracking not necessarily related to significant shearing of the supporting layers (base, subbase, and subgrade). While this did not directly impact on concepts of behavior of heavy duty airfield pavements, it did substantially influence the concerns and thinking of military (and most other) personnel responsible for airfield pavement design criteria.

Multiple-Wheel Heavy-Gear Load Failure

By the time the multiple-wheel heavy gear load pavement tests were conducted in 1970 and 1971, the responsible engineers were conditioned to the highway fatigue behavior. When the 12-wheel C-5 aircraft landing gear was applied to test pavements, the breadth of shallow depressions was so large that any effects of internal shearing was not observable from the surface and not readily apparent from measurements of layer thicknesses or densities in test pits. Failure was based on rutting depths and patterns.

Condition Ratings

Before closing these initial presentations on failure or terminal conditions, it should be pointed out that for the last 15 to 20 years it has not been in vogue to make specific failure determinations. This stems from the idea that a failed pavement should be unusable and from recognition that pavements, which have in the past been reported failed, have been continued in use. Deteriorated pavements are more likely to be termed severely distressed

than failed, and means have been devised for attempting to provide a measure of deterioration. For highways, the present serviceability index is used. For airfields, the pavement condition index has been formulated. These use observed and measured surface indications to arrive at a condition rating which can scale from excellent to failed. Ratings are numerical and are used to imply the degree or rate of deterioration. Some specific value can be selected to represent failure if such is pertinent.

CHAPTER 10

CONCEPTS HELD AND RESPONSES PROGRAMMED

Introduction

A great deal of respect must be accorded the original engineers who undertook the development of design criteria. The time proven validity of their early determinations and perceptions is quite remarkable.

At the outset they wanted to deal with the stress and strain induced in pavements by the loads imposed and then design pavement structures of sufficient strength to resist the stresses. The Westergaard equations offered promise of this for rigid pavements, but the engineers found no such equivalent method for flexible pavements. Recognizing the difficulties in developing theoretical means for representing behavior of flexible pavements, the engineers moved to select and adapt an established, experience based design method.

Stress Distribution Studies

Far from abandoning theoretical work, because hope for useful stress-strain based design was obviously too far in the future, a substantial program was undertaken to study the distribution of stresses in a pavement structure. Early accelerated traffic test sections at Stockton, California; Barksdale Field, Louisiana; and Marietta, Georgia included studies of induced stresses and deflections. Analyses were, however, severely limited by lack of any theoretical model of layered systems representing pavements. It was not possible to compare measured stresses to computed stresses which theory indicated should exist.

In the mid 1940's it was decided that since layered pavements could not be theoretically represented, test sections should be constructed to represent conditions which could be represented by developed theory. Accordingly, a test section was constructed of a clayey-silt soil and made to be as homogeneous and isotopic as possible. Loading plates were developed using a flexible face and water filling to permit application of a uniform pressure over a circular area. For these conditions the stresses, strains, and deflections could be directly determined from theory for certain special cases (under center of load, under edge of load, at the surface) and at any depth or offset position from Newmark's charts.

Pressure cells were installed in an array of orientations sufficient to completely define the stresses at each point, and deflection gages were emplaced to measure vertical deflection. Various magnitudes of load were employed for each of various size loading plates and for single loads and twin loads at several spacings. The loading program provided measurements at a number of depths and for a number of offset positions for each depth. From these, direct comparisons could be made between measured and theoretically determined values of stress and deflection.

A second test section of homogeneous sand followed in 1950-1951. Analysis and reporting took another 3 years. This was similar to the clayey-silt section but used a significantly different soil and an improved pressure cell.

It was a practice to use consultant boards to help plan and guide the research. These were composed of recognized experts. The board guiding the stress distribution studies was composed of the following:

- Dr. D. M. Burmister
- Dr. Arthur Casagrande
- Dr. M. Juul Hvorslev
- Dr. N. M. Newmark
- Dr. Gerald Pickett
- Dr. P. C. Rutledge
- Dr. D. W. Taylor
- Mr. T. A. Middlebrooks
- Mr. R. R. Philippe

Two-Layer Model

Dr. Burmister's two-layer analytical model had become available and a two-layer test section was planned. It was delayed pending improvement in understanding of pressure cell behavior (over-registration and pocket-action). However, shifting research emphasis along with reduced funding never permitted development of the two-layer test section.

Westergaard Model

For rigid pavements, the Westergaard equations (centerload) provided a theoretical model from which stresses and strains could be calculated. While there were no direct measurements for comparison, the pattern reflected by these equations did provide a workable basis for development of a design method. There was tacit acceptance that the model reflected actual stress and strain sufficiently for the purpose. There was no model for interface friction with contraction, for slab warping, or for repeated loading (fatigue) which required empirical factoring (design factor).

Rigid Pavement Behavior Concept

Rigid pavement is considered to distribute wheel loads in flexure or bending (beam action), and to receive support proportional to the vertical

displacement of the underlying surface (by Westergaard assumptions) of subgrade, subbase, or base. Support by the underlying surface (the proportionality) is rated in terms of the coefficient of subgrade reaction k and is expressed in pounds per square inch per inch of displacement or merely pounds per cubic inch. Slab capacity is limited by flexural stress. Ultimate flexural stress, as shown by representative beam specimens tested to failure, is reduced by a design factor to provide for unknown effects and for the magnitude of load (or stress) repetitions to be expected. The design factor is experience based and is varied to accommodate specific requirements.

Edge Load Equations

Westergaard edge-load equations became available in the mid 1940's and were adopted for Corps use for heavy airfield pavements. This brought about the question of load transfer at joints whereby the 25 percent factor was adopted. This factor was chosen from test results and experience and continues in use.

Flexible Pavement Behavior Concept

Flexible pavements were considered to be capable of compliance to underlying support provided on a layer by layer basis. They neither bridged across voids nor provided cantilever-like distribution in flexure. The tire load on the surface of a flexible pavement was considered to broaden and spread over a wider area at the bottom of the surfacing layer. This action obviously resulted in a reduction of the intensity of load or of the stresses. This same pattern was repeated through each successive lower layer of the pavement structure and into the subgrade.

Each layer needed sufficient resistance to the stress it must sustain. The structure must be thick enough to cause reduction in maximum stress to a level that the in-situ subgrade could sustain. For any layer or the subgrade to yield under the load (the stress level), it must shear so that resistance to shearing was the intent of design. The CBR test was considered to be a shear test and to represent a measure of shear resistance.

CBR design curves were plots of CBR value versus the thickness of overlying structure. This is required to reduce the stresses induced by the surface loading to the shear stress equivalent to required shear strength. Such curves thus permitted determination of the strength of each layer of the structure, of the subgrade, and of any low strength layers which might exist below the subgrade surface. The design process normally proceeded, in some-

what opposite fashion, to determine the required thickness to be provided above the subgrade (or deeper soft layer) and, successively, the thickness above each subbase depending upon the strength (the CBR) of each. Thickness of bituminous surfacing was also considered to be indicated as that necessary to reduce stresses to levels sustainable by the base, but these thicknesses were generally established somewhat greater and experience based to provide for longevity and resistance to environmental conditions.

In Situ Strength and Seasonal Variation

There was much concern, at the outset, for properly rating the shear strength of subgrades, subbases, and bases. Early concepts, which more simplistically related shear stress to shear strength, considered that the design strength should be the lowest strength which might be present during the life of a pavement. It was thus desirable to devise means for producing test samples to represent the soils as they will exist in the pavement structure. It was also necessary to devise means for conditioning test specimens to represent the lowest strength of materials expected in the pavement structure.

The early moisture-density-strength studies performed for development of CBR test methods showed that the standard proctor (or AASHO) compaction test would not provide sufficient maximum density to represent conditions expected to obtain in high performance airfield pavements. The modified AASHO compaction test was then developed by the Corps and has become a widely used test. The CBR test methods study also showed that soil strength does not uniquely relate to a moisture-density condition as had earlier been considered by many to be likely. Two soil specimens at identical density and moisture content, but prepared at different initial conditions and then adjusted to identical conditions, can have quite different strengths.

It was necessary to prepare representative CBR test specimens at the moisture and density expected to be employed in construction. It became accepted practice to conduct a modified compaction test at several moisture contents, plot the results and determine maximum density and optimum moisture content. A specimen would be prepared at optimum moisture to 90 percent of maximum density and used for CBR determination. A more desirable method was the preparation of several specimens at various moisture contents compacted by modified effort, several at an intermediate effort, and several at standard effort. Each of these specimens would be subjected to CBR test. Results were then plotted to show the complete range of moisture, density, and strength

from which the representative CBR could then be selected and moisture content range for construction control chosen.

The soaked CBR test (4-day soaking submerged) was devised to represent minimum strength which is likely to exist in a pavement during its life. CBR test specimens are prepared as discussed above and subjected to soaking prior to testing for (CBR) shear strength. The strengths determined are used for design and considered to represent the poorest subgrade or subbase conditions to be anticipated.

The soaked CBR test was considered by many to be unduly severe, but subsequent field moisture studies showed that the test provides ratings about right or only slightly conservative except for quite arid climate conditions. In areas of deep water table and low rainfall a thickness adjustment was introduced. Studies by the Corps and others have shown that, except near the edges, the moisture under bituminous pavements tends to attain and retain a relatively high percent saturation (perhaps 93 to 97 percent). This occurs 1 to 3 years following construction and continues with small to no seasonal variation as long as the pavement is maintained to prevent severe leakage of surface water.

Because this stable moisture condition does not extend beyond about 10 to 15 ft from the pavement edge, most highway pavements do not enjoy significant moisture-stable center sections. This is one area of substantial difference in behavior between airfield and highway pavements. It can be a source of misconception for pavement personnel familiar with highways when faced with airfield pavement concerns.

Separate Density Requirements

When early studies showed that strength or shear resistance is not uniquely related to density of soil materials, it followed that pavement design to satisfactorily limit shear failure would not inherently provide sufficient density to limit undesirable further compaction by traffic on the surface. It is thus necessary to establish compaction requirements for the subgrade and each pavement layer in addition to strength requirements.

Plasticity Limit for Density Control

It was recognized that nonplastic or cohesionless materials more readily gain density under application of compaction effort than do materials that have some cohesion. As a result, the compaction requirements must be greater for cohesionless materials and must reach to greater depths. The original CBR

test development studies reported an indication that the division between cohesive and cohesionless soils, for compaction purposes, could be taken at about plasticity index = 2. The compaction requirements study made in the late 1950's attempted to define graduating requirements from PI = 0 into the higher PI ranges, but failing this, because of insufficient data, established the dividing point at PI = 0. Widely accepted practice for other applications than compaction had established a division between cohesive (plastic) and cohesionless (nonplastic) soils at PI = 6. In Corps work a value of PI = 5 for other applications is commonly found. Unfortunately in some compaction requirements applications of Corps study results by other agencies and in some Corps applications the PI = 6 (or 5) value has been erroneously introduced, and it continues in existing criteria.

This is not the insignificant matter it may first appear. The deep compaction requirements for noncohesive soils under heavy airfield pavements often cannot be attained from the subgrade surface. In such cases, in cut sections, it becomes necessary to excavate and recompact the subgrade soil to attain required density. This obviously is a costly process and to require such operations unnecessarily on soils whose PI is between 3 and 5 (or 6) is a poor practice.

Load Repetitions

The interdependence of load magnitude and load repetitions was not originally recognized. It was then considered that the greatest load a pavement could support for several thousand applications could continue to be supported for the life of the pavement. It was recognized that a pavement would not fail under a single application of a load slightly larger than one it could support.

In accelerated traffic testing it was accepted that repeated application of the test load was required, and that too large a load would cause pavement distress within a thousand to several thousand repetitions. A pavement satisfactory for a certain load would sustain 1,000 (to several) repeated applications of that load.

The narrow, small wander, wheel paths effective on highways were not the same for airfields. Even on taxiways that wheel path of an aircraft might be considerably offset from that followed on the next pass. It was reasoned that the pavement at any particular point could be considered to have sustained a load (repetition) if the point fell beneath a passing tire.

Traffic applied in accelerated traffic testing was arranged to fall in adjacent wheel paths to cover a width of tracking lane. Somewhat consistent with earthwork compaction using pneumatic-tired rollers, the wheel tracking to cover an area was termed a coverage. One load repetition was considered to be applied by one coverage in the lane being trafficked.

It was found that test traffic could not be all applied in a single-wheel path and considered to represent coverages where passes would equal coverages. Coverages for traffic distributed over three or more wheel paths are more severe than passes in a single-wheel path taken to represent coverages. While it might at first be reasoned that three passes in adjacent wheel paths should be expected to be more severe than single passes in a single-wheel path, it has been found that equal numbers of passes, whether distributed in three adjacent wheel paths or all in a single-wheel path, are more severe in the distributed application.

It was also learned that wheel passes equal to the full coverage level could not extend to the edge of a tracking lane in accelerated traffic testing when no passes were applied outside the tracking lane. Obviously, on active pavements the wheel passes would have a central tendency and would reduce with offset following some sort of bell-shaped curve. Full coverages would obtain only at the center of this repetition pattern. Accelerated test traffic has been programmed to apply full coverages over a central lane with successively less traffic in outer wheel paths.

While test traffic can be placed as planned, the actual traffic on airfield pavements cannot. Since wheel passes range (or wander) over a lane wider than one tire print width, not every pass contributes to the maximum central accumulation which represents coverages. The patterns of lateral distributions or wander of aircraft during operation on taxiways and runways have been studied and used to determine pass-per-coverage ratios for the various aircraft types. These ratios are used in translating actual traffic in terms of operations to coverages of traffic loads for use in design and evaluation of pavements. Taboza (1977) gives some explanation of pass-per-coverage ratios.

When dual wheels and dual tandem were introduced for aircraft landing gear, the pass-to-coverage patterns were affected. For duals, both tires contribute to coverages within the tracking lane, but with no overlap. For tires in tandem (one directly and closely following another), there is a ques-

tion of whether two passes both contribute fully toward coverages. It was decided, for flexible pavements, that tandem wheels would contribute two load repetitions and would both be counted toward coverages. For rigid pavements, however, it was considered that only one load pulse or repetition is delivered at the bottom of the slab, and the two wheels in tandem are considered to apply only one unit toward accumulation of coverages.

Pavements can be designed to sustain a selected load (aircraft type and weight) for a projected number of repetitions, and an established loading can be applied to an accelerated traffic test section, but actual loads experienced on in-service pavements vary significantly in character and magnitude. It is therefore necessary to provide for a mixture of loads for design and particularly for evaluation of pavements. This has been done by devising means for reducing an array of loadings to an equivalent in terms of repetitions of a selected basic or critical load.

The process is one of assigning (determining) an equivalent number of repetitions of the chosen basic load to each load actually sustained or expected to be applied to the pavement. In concept this involves determining the repetitions of each load which could be sustained by a pavement designed for the basic load and dividing these repetitions by the basic load repetitions used for the design.

The requirement for structural capacity of a pavement, as represented by repetitions of a demanding load, has been found to relate as the logarithm of the number of repetitions. Thus, it is not a particular number of repetitions so much as the cumulative level or order of magnitude of repetitions applied to a pavement that is significant. For example, doubling repetitions represents only a small percentage (perhaps 7 percent for flexible pavements) increase in structural requirements. So the treatment of load repetitions in pavement design is a forgiving process, and good precision is not required.

This is both a great advantage and a severe constraint. It is an advantage because only very crude methods can be satisfactorily applied to traffic and loading inputs to pavement design. It is a severe constraint in that it permits continued use of methods which only a cursory examination will show are crude but which would require an inordinate effort to improve.

Some of the crudities practiced are explained below.

a. The diverse tread spacing on various type aircraft place the wheels in different offset positions such that wheel passes can be in separate locations for two different aircraft. With very few exceptions (Asphalt Institute, MS-11 is one), the pattern of wheel passes for different aircraft types, even including bicycle gear aircraft, have been directly combined as though all such patterns were aligned.

b. The coverage concept, which only combines wheel passes that overlap one another, does not account for the obvious overlap of the spreading load at depths within a pavement.

c. For combining the effects of mixed type and magnitude of loads equivalencies of one load to another have been employed. Commonly, these equivalencies are determined by comparing the evaluated total repetitions for the two loadings being compared for a selected pavement structure. The comparison is by simple ratio. It is well established that structure capacity relates to load repetitions logarithmically. Thus, any simple ratio will be different for different magnitudes of load repetitions so that the adopted practice is arbitrary and unverified. These comparisons can be interpreted to imply, with no proof for or against, that the load equivalencies are not single valued but may vary widely for newer (low repetitions) to older (high repetitions) pavements.

d. There may be different cumulative repetitions effects for two single wheels passing at the same lateral spacing as a dual and the two wheels of the dual. The same question applies to two singles in the same wheel path at some interval and two wheels closely in tandem. As earlier noted, wheels in tandem are not treated the same for flexible and rigid pavements.

Drainage Effectiveness

It has been common practice to place great emphasis on drainage. This implies that where pavements can be properly drained less pavement structure will be required. The Corps practices soaked CBR and the field moisture studies, which showed the soaked determinations not to be unduly conservative, appear to convey a concept that drainage is not of much advantage in reducing the pavement structure necessary. It has been found that, for all but the edges of wide pavements, the in-place materials (subgrade and subbase) will attain a near saturated condition, and drainage cannot be depended upon to prevent this problem. It does not, however, follow that drainage is

ineffective and not required. The removal of gravity water and avoidance of complete saturation with attendant pore pressure conditions are of such significance that the drainage requirements are clearly justified.

Pavement Pumping

Initially and for some years, it was the general consensus that military airfield rigid pavements did not suffer from pumping. This consensus continued despite recognition of pumping as a severe problem for rigid highway pavements both in service and in the various test sections in the 1950's and 1960's.

The highway practice of specifying filter layers of granular material beneath rigid pavement to minimize pumping was not considered necessary or particularly desirable beneath rigid pavement for airfields. Since then, it has been considered that use of base or subbase under rigid airfield pavements should depend on design economics. Base layers were neither avoided nor preferred in the earlier period. The advent of channelized traffic and attendant experience of some limited pumping in both test pavements and in-service pavements have resulted in some modification of early thinking.

It should perhaps be noted that there was not any clear preference for terming a granular layer beneath rigid pavement as base or subbase. It has had no serious consequences, but both terms are commonly applied, generally with no particular differentiation.

The current preference for some agencies, most notably the FAA, to require a stabilized base (or subbase) beneath rigid pavements has not found endorsement among Corps pavement engineers.

Measure of Use-Life

Before it was clearly recognized that a pavement has a load capacity related to load repetitions, it was readily accepted that design or evaluation was a simple process of relating structure to load to be supported. Since the role of load repetitions, in relation to load magnitude, has been strongly established, there has been a clear and strong need for means to measure the remaining life of in-service pavements.

Design undertakes to provide a pavement capable of sustaining an established design load for the number of repetitions compatible with a use-life period commonly considered to be about 15 to 20 years. If a pavement has been designed for 20-years life and has been used as intended for 10 years, it should have a residual life of 10 years.

There is a strong desire to have some measurement process capable of being applied to in-service pavement to indicate its remaining use-life. The means for such measurement is commonly falsely assumed and evaluations too frequently misinterpreted.

The present serviceability index (PSI) devised for the AASHO road test analysis was an attempt to formulate a use-life measure. The index has been widely employed but is not very satisfactory, except in concept. The pavement condition index (PCI) has been similarly devised as a measure of use-life for airfield pavements. The PCI is becoming widely, though not universally, accepted. It is satisfying a great need and is easily accepted in concept. It does not have good verification but it does have great reservation by many.

Pavements which are not severely overloaded do not evidence significant deterioration or distress until they have entered a terminal condition or failing mode. This occurs only later in the use-life of a pavement. Where weak subgrades are involved, this failing mode can be quite short in transition from early indications to severe distress. For strong subgrades, the transition from initial indications of distress to severe or terminal conditions can be rather protracted. For these, the PSI and PCI can provide indications of increments of use-life expended, but only after the pavement has entered a terminal condition or failing mode. For earlier usage, these indexes should show no change.

Direct physical measurements employed, such as deflection under load and strain magnitude, tend to provide the same indication (magnitude of measurement) between cure-out following construction and entering a terminal deterioration condition.

Multiple-Wheel Concepts

When the B-29 and B-36 introduced twin and twin-tandem (multiple-wheel) landing gear configurations late in the 1940's, the empirical nature of flexible pavement design did not permit extrapolation to multiple loads. Since it did not appear feasible to directly generate experience-based relations for multiple-wheel loads, the concept of equivalent single-wheel load was introduced. By generating means for relating an equivalent single-wheel load to a specific (dual and dual-tandem) multiple-wheel loading, the already well established single-wheel design criterion was made to serve for multiple wheels.

Nondestructive Testing of Pavements

The process of evaluating the load support capacity of pavements by use of empirical relations between deflection under load and load-support capability was employed quite early by some pavement experts. In some respects it became almost a routine with the devising of the Benkelman Beam used on the WASHO and AASHO road tests. But the nondestructive testing terminology and the related methods really began with devices developed to dynamically load pavements with steady-state inertial loadings, mostly sinesoidal, in the early 1960's or perhaps a bit earlier.

The process perhaps related more to a developed capability to measure the responding pavement deflection with seismic-type inertial transducers. Both instrument and computer developments contributed to a capability to measure velocity or acceleration and reduce the measurements to displacements.

Early efforts attempted to evaluate load support capacity from maximum load displacement (deflection), from offset displacements and deflection basin shape, and from various aspects of wave-velocity measurements.

These efforts matured into the dynamic stiffness modulus methods used by the Corps, the Air Force, the FAA, and the ICAO. Some wave-velocity methods continued to be developed by the US Air Force.

At present, the steady-state vibratory loading methods have largely been supplanted by falling weight deflectometers which can induce loads of wheel-load magnitude while being much more readily transportable.

Mathematical and computer developments permit back calculation of effective moduli of pavement layers for models simulating the pavements. Thus, stresses and strains introduced can be calculated for use in establishing limiting loadings (pavement evaluation).

Concepts Held and Responses Programmed

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